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16. ABSTRACT

As part of joint interests in environmental matters between the United States and Japan, a Conference on Sewage Treatment Technology is held at intervals of about 18 month. This pbulication contains papers from the Japanese group and from the American side that were presented at the Seventh Conference held in Tokyo, Japan. Subject matter covered included innovative & alternative technology, regional approaches, toxic wastes, health effects, combined sewer technology and reuse of wastewater.

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December 1980

PROCEEDINGS

SEVENTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

MAY 19-21, 1980
TOKYO, JAPAN

OFFICE OF INTERNATIONAL ACTIVITIES
OFFICE OF WATER AND WASTE MANAGEMENT
WASHINGTON, D.C. 20460

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FOREWORD

Environmental improvement is a worldwide need. Maintaining clean water supplies and managing municipal and industrial wastes is a vital element of a quality environment.

The participants in the United States-Japan cooperative project on sewage treatment technology have completed their seventh conference. These conferences, held at 18-month intervals, give the scientists and engineers of the cooperating agencies an opportunity to study and compare the latest practices and developments in the United States and Japan. These Proceedings of the Seventh Conference comprise a useful body of knowledge on sewage treatment, which will be available not only to Japan and the United States but also to all nations of the world who desire it.

Douglas M. Costle
Administrator

Washington, D.C.

CONTENTS

FOREWORD.................................. iii

JAPANESE DELEGATION....................... vi

UNITED STATES DELEGATION.................. vii

JOINT COMMUNIQUE.......................... 1

JAPANESE PAPERS........................... 5

UNITED STATES PAPERS...................... 365

TECHNICAL SEMINAR PAPERS (U.S.)........... 719

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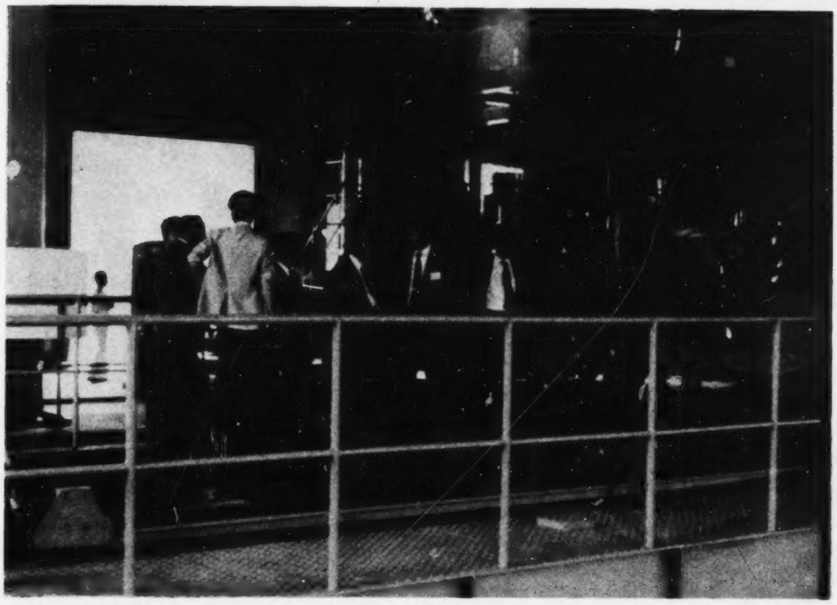
DELEGATES ON FIELD VISIT



DELEGATES VISIT "WITHIN-VESSEL" COMPOSTING FACILITY
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JOINT COMMUNIQUE

SEVENTH UNITED STATES/JAPAN CONFERENCE ON SEWAGE TREATMENT TECHNOLOGY

TOKYO, JAPAN
MAY 21, 1980

1. The Seventh United States/Japan Conference on Sewage Treatment Technology was held in Tokyo from May 19 to 21, 1980.

2. The United States Delegation headed by Mr. John J. Convery, Director, Wastewater Research Division, Municipal Environmental Research Laboratory, U.S. Environmental Protection Agency (USEPA), Cincinnati, Ohio, was composed of three representatives from the USEPA, one representative from the State of Florida, and one representative from a local authority (Los Angeles County).

3. Mr. Satoru Tohyama, Director, Department of Sewerage and Sewage Purification, Ministry of Construction, was Head of the Japanese Delegation, which consisted of five National Government representatives, three Japan Sewage Works Agency representatives, and three local government officials (Tokyo, Kyoto, Yokohama).

4. The chairmanship for the Conference was shared jointly by Mr. John J. Convery and Dr. Takeshi Kubo, President of the Japan Sewage Works Agency.

5. During the Conference, papers relating to joint research projects on sludge treatment/disposal and instrumentation, and automated control of municipal sludge treatment facilities were presented by both sides. A Progress Report on U.S./Japan Joint Research Projects (Japanese side) was presented. Data and findings on the joint research projects were useful to the development of improved control technology practices for each country. A decision was made to expand the scope of the joint research projects conducted by the two countries to include energy conservation, operation and maintenance practices, and small flow treatment systems.

6. Principal topics of the Conference were current water pollution control programs in each country including innovative and alternative technology, regional approaches, toxic wastes, health effects, combined sewer technology and reuse of wastewater. The discussions which followed each presentation were mutually productive and useful to both countries.

7. In addition to the Conference, the technical seminar organized by the Japan Sewage Works Association is planned, and about 200 active members of the Japan Sewage Works Association are registered to attend the seminar. During the seminar, presentations will be made by each U.S. delegate to be followed by floor discussions.

8. Field visits in Kyoto, Himeji, Tokyo, Yokohama, and Tsukuba areas are planned to inspect wastewater treatment facilities in these areas.

9. Recent engineers exchanges between the two countries included a two-week visit in 1979 to Japan by Mr. James C. Gratteau, County Sanitation Districts of Los Angeles County, California, and a six-month long visit in 1979 to the United States by Dr. T. Mori, Japan Sewage Works Agency. Mr. S. Kyosai, Public Works Research Institute, Ministry of Construction, is spending 12 months in the United States. Both sides agreed to continue the engineers exchange programs.

10. A new initiative was agreed to, which will include the exchange of summaries of important research reports and design guidelines on new technologies.

11. It was proposed by the United States side that the Eighth Conference shall be held in the United States about October 1981.

12. A Proceedings of the Conference will be printed in English and Japanese.

JAPANESE PAPERS

BASIC POLICIES FOR SEWAGE WORKS IN JAPAN........................... 5
Satoru Tohyama, Director, Department of Sewerage
and Sewage Purification, Ministry of Construction

CURRENT ISSUES IN WATER POLLUTION CONTROL ADMINISTRATION IN JAPAN.. 37
Toshiki Oshio, Environment Agency, Government of Japan

REGIONAL WASTEWATER TREATMENT PLANNING IN TOKYO BAY AREA........... 57
Tsutomu Tamaki, Head, Sewage Works Division, Department of
Sewerage and Sewage Purification, Ministry of Construction

AUTOMATED CONTROL OF SLUDGE TREATMENT SYSTEMS...................... 89
Mamoru Kashiwaya, Kinichiro Azuma and Akio Kuwayama,
Research and Technology Development Division,
Japan Sewage Works Agency

ENGINEERING EVALUATION OF MUNICIPAL SLUDGE INCINERATORS............ 121
Kazuo Ohmiya and Sadaharu Takahashi, Research and Technology
Division, Japan Sewage Works Agency

PUBLICLY OWNED TREATMENT WORKS PRETREATMENT CONTROL PRACTICE
IN TOKYO.. 167
Tetsuichi Nonaka, Senior Technical Advisor,
Sewage Works Bureau, Tokyo Metropolitan Government

DEODORIZATION IN SEWAGE TREATMENT PLANTS OF YOKOHAMA............... 199
Shigeki Miyakoshi, Head, Construction Division,
Sewage Works Bureau, City of Yokohama

AUTOMATIC WATER QUALITY ANALYZERS FOR SEWERAGE SYSTEMS............. 243
Ken Murakami, Public Works Research Institute,
Ministry of Construction

CURRENT STATUS OF COMBINED SEWER PROBLEMS AND THEIR CONTROL
MEASURES IN JAPAN... 263
Muneto Kuribayashi, Director, Water Quality Control Division and
Eiichi Nakamura, Research Engineer, Sewage Works Section, Public
Works Research Institute, Ministry of Construction

WATER QUALITY IMPROVEMENT IN YODO RIVER AND SEWAGE WORKS IN KYOTO.. 311
Takashi Yoneda, Director, Sewage Works Bureau, City of Kyoto

PRACTICAL APPLICATIONS FOR REUSE OF WASTEWATER..................... 331
Takeshi Kubo, Vice President, Japan Sewage Works Agency

Seventh US/Japan Conference
on
Sewage Treatment Technology

BASIC POLICIES FOR SEWAGE WORKS IN JAPAN

May 19, 1980

Tokyo, Japan

Satoru Tohyama

Director,

Department of Sewerage and Sewage Purification,

Ministry of Construction

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BASIC POLICIES FOR SEWAGE WORKS IN JAPAN

1. Present Situation of Sewerage Installations in Japan 9

2. Role of Sewerage and Its Diversification 12

3. Conservation of Water Quality as Water Resources 13

4. Promoting Effective Utilization of Resources 16

5. New Developments in Sewerage Administrative and Financial Policies 18

5.1 Sewerage Improvement 18

5.1.1 Targets for Improvement 18

(a) Basic Policies 18

(b) Long-term Arrangement Targets 20

(c) Immediate Arrangement Targets 21

5.1.2 Method of Sewerage Improvement and Arrangement 22

(a) Effective Promotion of Sewerage Arrangement .. 22

(b) Promotion of Tertiary Treatment 23

(c) Promotion of Sewerage Arrangement as Flood Control Measures 25

(d) Harmony with Community 26

(e) Promotion of Energy and Resource Conservation 27

5.1.3 Securing Financial Resources for Sewerage Arrangement 28

5.2 Maintenance and Management of Sewerage 29

5.2.1 Methods of Maintenance and Management 29

(a) Wide-scale Disposal of Sewage Sludge 29

(b) Strengthening of Control over Pre-treatment Facilities 31

(c) Promotion of Effective Use of Resources 32

(1) Effective Utilization of Sewage Sludge 32

(2) Reutilization of Effluent from Sewerage 33

(d) Arrangement of Executing System 34

5.2.2 Securing Financial Sources for Maintenance and Management 35

Thus, it may be said that sewerage installations in this country are presently in the "developing stage". It cannot be denied that this has occurred because the importance of sewerage service has not been fully recognized by the people and because Japan has been backwardness in urban development. Urbanization has begun without providing sewerage services.

However, people's requests for sewerage systems have greatly increased in recent years and, in response to this, the government has also expanded its sewage works recently.

Real sewerage layout in this country began after the enactment of the Sewerage Law in 1958 and the First 5-year Sewerage Arrangement Plan in 1963. A 5-year Sewerage Arrangement Plan, which is the basic plan for sewerage improvement, has been executed 4 times from 1963 to the present. The project scale has tripled at each time of the plan revision. Economic plans have been made several times up to now, and the position of sewerage investment within public investment has rapidly improved. That is, the share of sewerage investment was only 3.3% in the Medium Term Economic Plan from FY 1958 to FY 1968, but it reached 7.1% in the Economic Plan for the Second Half of the 1970's, which means that share of sewerage investment doubled during this period and the amount of investment increased about 12 times. In addition, according to New Economic and Social Seven-Year Plan, established in August last year, public investment amounting to 240 trillion yen (prices as of 1978) was planned for the 7 years from FY 1979 to FY 1985.

A sum of 18.2 trillion yen was appropriated for sewerage services. Its share of total investment was 7.58%, a further increase.

BASIC POLICIES FOR SEWAGE WORKS IN JAPAN

1. PRESENT SITUATION OF SEWERAGE INSTALLATIONS IN JAPAN

In recent years although priority in investment has been placed on the projected upgrading of social overhead capital to improve the quality of people's lives, the present status of diffusion of sewage coverage in Japan is not necessarily satisfactory.

With respect to the rate of diffusion of sewage coverage at the end of fiscal year 1978, population in the areas where sewerage services were available was only 27% of the whole population in this country. This means that two-thirds of the population was not receiving sewerage services..

On the other hand, the diffusion of water supply systems has already reached 90% in this country and, thus, there is a large gap between the diffusion of sewerage and water supply systems.

With respect to the situation of diffusion in urban areas, the mean rate of diffusion in the ten largest cities at the end of fiscal year 1978 was 66%, while that of other areas was only 15%, indicating a presence of unbalanced sewerage diffusion that varies depending upon the urban scale and locality.
(Refer to Fig. 1)

Fig. 1

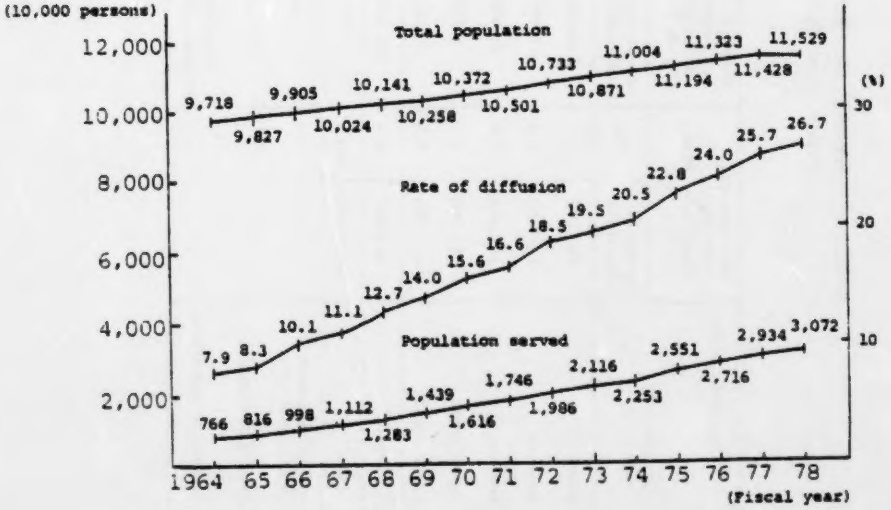


Table-1 Amount of Social Overhead Capital Investment under Economic Plans and 5-year Plans for Sewerage Arrangement

(Unit: 100 million yen)

Economic plans / Public works	New Economic and Social Seven-Year Plan ('79 ∿ '85)		Economic Plan for the Second Half of the 1970's ('76 ∿ '80)		Basic Economic and Social Plan ('73 ∿ '77)		New Economic and Social Development Plan ('71 ∿ '75)		Economic and Social Development Plan ('67 ∿ '71)		Medium Term Economic Plan ('64 ∿ '68)	
	Amount of investment	Share	Amount of investment	Share	Amount of investment	Share	Amount of investment	Share	Amount of investment	Share	Amount of investment	Share
Roads	460,000	19.2	195,000	19.5	190,000	21.1	117,000	21.3	61,500	22.4	41,000	23.0
Housing	135,000	5.6	65,000	6.5	60,800	6.8	39,000	7.1	17,100	6.2	11,200	6.3
Sewerage	182,000	7.6	71,000	7.1	56,500	6.3	23,000	4.2	9,300	3.4	5,792	3.3
Urban parks	45,000	1.9	15,400	1.5	13,000	1.4	4,300	0.8	2,070	0.7	805	0.4
Water supply	142,000	5.9	55,000	5.5	47,000	5.2	29,000	5.3	16,100	5.9	9,000	5.1
Total	964,000	40.2	401,400	40.1	367,300	40.8	212,300	38.6	106,070	38.6	67,797	38.1
Gross	2,400,000	100	1,000,000	100	900,000	100	550,000	100	275,000	100	178,000	100
5-year Sewerage arrangement plan			4th				3rd		2nd		1st	
Period of project			'76 ∿ '80				'71 ∿ '75		'67 ∿ '71		'63 ∿ '67	
Amount of investment			(Contingency) 71,000+4,000=75,000				(Contingency) 25,000+1,000=26,000		(Contingency) 9,000+300=9,300		4,400	

With respect to the forth 5-year Plan for Sewerage Arrangement which is currently being carried out, the period of this plan is from FY 1976 to FY1980 and a total amount of 7.5 trillion yen was appropriated for construction. Works amounting to 1.7816 trillion yen are being carried out in FY 1980 as the last year of this plan. As a consequence, the completion rate of the 5-year plan will 98.4% and sewerage diffusion to population served will finally become 30%.

On the other hand, with respect to the status of the construction of sewerage works, local public bodies are very aggressive in carrying out these projects because more people now recognize the importance of sewerage facilities. At the end of FY 1978, public sewerage construction work was being carried out in 616 cities and towns, 19% of the total number (3,255) of cities and towns in this country. Treatment was started in 291 cities and towns, 9% of the total. (Refer to Table-2)

Table-2 Completion Stage of Public Sewerages

(As of March 31, 1979)

Population class	Less than 100×10^3	100×10^3 to 300×10^3	300×10^3 to 1×10^6	More than 1×10^6	Total
Total number of cities & towns	3,082	126	39	10	3,257
Number of cities & towns undertaking projects	448 (15%)	119 (94%)	39 (100%)	10 (100%)	616
Number of cities & towns in which treatment has begun	143 (5%)	101 (80%)	37 (95%)	10 (100%)	291

With respect to River Basin sewerage, operation was already started at 24 out of 66 places. It may be said that the "construction-maintenance-management age", where both construction and management will be performed together, has just begun in this country.

2. ROLE OF SEWERAGE AND ITS DIVERSIFICATION

The role of sewerage treatment has expanded as times change. When modern sewerage treatment was first introduced, its main purpose was to improve the living environment by immediately removing sewage and to prevent floods by removing storm water, today sewer treatment is performed by sewerage facilities so that it now has become an indispensable service for preventing water pollution in public water areas. In addition, as a result of improvements in the standard of living and modes of life of people in recent years, they now are more concerned with bettering the quality of life. Thus, people now think that not only those who live in urban areas but also those living in farming areas naturally have the right to enjoy sewerage services.

Also, special attention must be paid to the fact that sewerage systems are more frequently being built as part of the national land policies of the government. The Third Comprehensive National Development Plan established in November, 1977 had adopted an "Integrated Residence Policy" as a basic planning approach in its national land policies for the future. This scheme is intended to form an all-round environment for human occupancy where the natural, human and production environments would be harmonized together. In order to realize a population arrangement reflecting this policy (Table-3) and particularly to promote stable living conditions for the young generation in local areas, it is required to preferentially carry out sewerage arrangement, an important factor for the formation of an all-round living environment in rural districts.

Table-3 Population Frames for Integrated Residence by the Third Comprehensive National Development Plan

District	Actual population (unit in 10,000)			Increase in population (unit in 10,000)	
	1975	1985	2000	1975∿1985	1975∿2000
Tokyo Metropolitan Zone + Osaka Zone	4,270 (3,520)	4,850 (4,260)	5,380 (5,080)	580 (740)	1,110 (1,560)
Others	6,920 (2,860)	7,780 (3,880)	8,860 (5,100)	860 (1,020)	1,940 (2,240)
Total	11,190 (6,380)	12,630 (8,140)	14,240 (10,180)	1,440 (1,760)	3,050 (3,800)

Note: DID (Densely Inhabited District) population frame is shown in ( ).

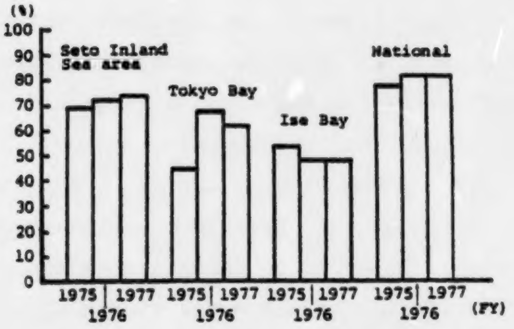
3. CONSERVATION OF WATER QUALITY AS WATER RESOURCES

In general, water pollution is being managed better as a result of a strengthening of control in the quality of effluent and sewerage arrangements in recent years. Particularly, sewerage treatment is performing a very important role in improving the water quality of urban rivers. For instance, water quality is improving very quickly as sewerage improvement work progresses in the basins of rivers running through large cities, such as the Sumida River in Tokyo, and through major regional cities such as the City of Sendai and the City of Okayama. For example, sweetfish and bull trout, which usually live in mountain streams, now can be caught in the Hirose River which runs through the City of Sendai. Also, in the City of Sapporo where the winter olympics were once held, salmon now come up the rivers which run through this city. In the Sumida River of Tokyo, a traditional boat race between Waseda University and Keio University and fireworks festivals were discontinued for many years because of pollution. They are now reinstituted and are again summer attractions in Tokyo. All of this clearly indicates the positive effects of the sewerage treatment improvements.

With respect to all public water areas, only 0.07% of all water areas contain materials harmful to human health, such as cadmium, that do not conform to the environmental quality standards of FY 1978. The number of water areas having harmful materials not conforming to this standard is decreasing each year.

On the other hand, in view of the degree of achievement in environmental water quality standards for "living environment items", 21.2% of rivers, 35.6% of lakes and 17.5% of sea areas do not conform to the values of environmental quality standards in FY 1978. These circumstances are undesirable. Particularly in wide, closed-type water bodies such as Tokyo Bay, Ise Bay, and the Inland Sea are, a rich nutriment of water has occurred as a result of the accumulation of nutritious salts such as phosphorus which are contained in residential and industrial waste water due to a poor exchange of water. Organic material such as plant-type plankton has increased, which cause red tides each year and damages to culture-type fisheries. (Refer to Fig. 2 and Table-4)

Fig. 2 Degrees of Achievement of Environmental Water Quality Standards in Wide, Closed-type Water Bodies



Remarks: 1. This is the rate of achievement related to COD value.

2. Rate of achievement is derived by

$$\frac{(\text{No. of water areas where env. quality stds. are achieved})}{(\text{No. of water areas where env. quality stds. are met})} \times 100(\%)$$

3. "National" means water areas other than Tokyo Bay, Ise Bay and the Inland Sea.

(Source: Environment White Paper, FY 1978)

Table-4 Number of Confirmed Occurrences of Red Tide

Water area / Year	Tokyo Bay	Ise Bay	Seto Inland Sea
1971		55	108
1972	3	47	140
1973		56	161
1974	3	36	113
1975	3	22	224
1976	7	33	148
1977	13	35	125
1978	22	31	74

Source: "Present situation of Ocean Pollution" surveyed by Maritime Safety Agency

Therefore, for the water areas stated above, a total mass effluent control system that aims to reduce the total quantity of pollution was introduced in June last year in addition to the conventional concentration control. This aims to reduce the present total quantity of pollution by approximately 10% by 1984, the target year. Sewerage treatment is considered to be an important measure governing the success of this total mass effluent control system. Quantitative and qualitative upgrading of sewerage treatment is urgently sought for.

In addition, some water areas now require the use of tertiary treatment to improve the rate of nitrogen and phosphorus removal in order to prevent excessive nutriment and to further improve the quality of water normally subjected to secondary treatment for removing BOD, SS, etc.

"Present Situation of Execution of Tertiary Treatment Work"

Place:	Executing body:	Start of work (FY):	Start of treatment (FY):	Method of treatment:
[River basin Sewerage]				
Tama River catchment sewerage: Minamitama Treatment Plant	Metro. Tokyo	FY 1974	FY 1975	Concentrated settling + quick filtering (partly including absorption by activated carbon)
Kasumigaura Jonan catchment sewerage: Tone Purification Center	Ibaraki Pref.	FY 1973	FY 1978	Concentrated settling + quick filtering
Teganuma catchment sewerage: Teganuma Sewage Treatment Plant	Chiba Pref.	FY 1978	FY 1981 (planned)	Quick filtering
[Public Sewerage]				
Ochiai Treatment Plant	Metro. Tokyo	FY 1976	--------	Concentrated settling + quick filtering
Hirano Treatment Plant	Osaka City	FY 1976	--------	Quick filtering
Tamatsu Treatment Plant	Kobe City	FY 1980 (planned)		Concentrated settling + quick filtering

4. PROMOTING EFFECTIVE UTILIZATION OF RESOURCES

According to "the Water Resources Development Plan and Water Utilization toward 1990" (Ministry of Construction) announced in November, 1978, an insufficient supply of water in the future is forecasted. For instance, a shortage of 6.91×10^8 m^3/year in the Minami-Kanto District, of 3.01×10^8 m^3/year in the Keihanshin

District, of 2.22×10^8 m^3/year in the Northern Kyushu District, and of 15.0×10^8 m^3/year in total for ten other districts throughout the nation is forecast, so that securing new water resources has now become a big problem that must be solved.

As a matter of fact, a serious water shortage occurred in the City of Fukuoka in the northern district of Kyushu during the summer two years ago, thus revealing the weakness of large cities to water shortages. In addition to the City of Fukuoka, there are many other districts and cities where serious water shortages similar to the one in the City of Fukuoka could occur at any time; some of these areas are the South Kanto, and the cities of Sendai, Nagoya, Kyoto and Kobe.

In order to cope with such circumstances, the reutilization of treated water from sewerage has been more frequently reviewed or reexamined in recent years as part of the effective utilization measures for water resources.

At present, about 60×10^8 m^3 of treated water is discharged from sewerages each year. But only 3×10^8 m^3/year is being used as cooling water and cleaning water at final sewage treatment plants, and about 1×10^8 m^3/year is being used as industrial water and landscape water outside of the treatment plants. The remaining water is directly discharged to public water areas at present. In parallel with progress in sewage works in future, it is expected that treated water from the sewarages will continuously increase in quantity (98×10^8 m^3/year with a rate of diffusion of 40%; 180×10^8 m^3/year with 70%; and 255×10^8 m^3/year with 90%) and thus it is extremely important to reexamine and utilize, as precious water resources, a tremendous amount of treated water from the sewerages.

However, it is very significant that the City of Fukuoka began a model project from FY 1979 which will supply treated water from the sewage treatment plants to other public facilities as water to be used for toilets.

In addition, utilization of sewage sludge from sewage treatment as a new resource is an important problem to be coped with. The amount of sewage sludge generated in FY 1976 amounted to 2.4×10^6 m^3/year (70% water content), but only about 10% of this amount was effectively used for recovery by green farm lands, and about 80% was disposed in land and sea reclamation areas and a remaining 10% returned to the ocean. It is expected that the amount of sewage sludge generated will increase further as sewage works progress in future (11.7×10^6 m^3 in FY 1985 with 70% water content). However, securing lands that can be reclaimed in this country is getting extremely difficult, and sewage sludge can be disposed of in such reclamation lands in the nation only for the next 10 years in view of the present situation of reclamation land acquisition. Some cities have enough land for only the next year or less for sludge disposal.

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In improving the sewerages for the future, disposal of the sewage sludge itself is an important problem and it seems necessary to promote the treatment and disposal of sewage sludge in wider areas but, at the same time, it is considered necessary to aggressively carry out the effective utilization of sewage sludge such as recovery for green farming lands with special attention to the usefulness of sludge as one of the resources.

5. NEW DEVELOPMENTS IN SEWERAGE ADMINISTRATIVE AND FINANCIAL POLICIES

5.1 Sewerage Improvement

5.1.1 Targets for Improvement

(a) Basic Policies

It is apparent that sewerages must be so improved that they may be able to appropriately resopond to many social needs, and the consensus of the majority in this country is that top priority in the execution of sewage works should be given to the improvement of the rate of diffusion of sewerages for the time being in order to quickly cope with the considerable slowness in sewerage arrangement.

Also, it is very clear that local public organs that are in the stage of maintenance and operation as a result of progress made in the sewage works, must properly maintain and operate the sewerages in order to effectively provide for the functions of sewerages.

Under the present circumstances in which the Japanese economy is shifting toward long-term stable growth in response to the changes in the domestic and foreign environment, such as severer restrictions on use of energy and resources, integrated policies such as the expansion of measures against pollution sources for preventing water pollution, must be appropriately carried out in order to adequately and totally promote the formation of a better living environment. Special precautions must also be taken for the following points when improving sewerages as basic infrastructure for the environment:

(1) To steadily improve the rate of diffusion of sewerage treatment, sewerage improvement in live with local needs is to be promoted; for example, sewage treatment methods and maintenance and operation measures are to be selected by fully considering the actual local situation. If necessary, quality improvement is to be undertaken. For example, the upgrading of existing facilities, and promotion of tertiary treatment for preventing excessive nutrition are to be made.

(2) To maintain the function and stable service of sewerages, proper administraton and operation are to followed. For example, the proper management of sewage facilities, proper treatment and disposal of sewage sludge, and the strengthening of control and monitoring of inferior wastewater will be necessary.

(3) To promote the sound development of sewage works, adequate allocation principles for the burden to local public organs and users with respect to construction, maintenance and operation of sewage facilities are to be established, and a financial base for the sewage works is to be strengthened.

(b) Long-term Arrangement Targets

The following targets for long-term sewerage arrangement are to be established toward the year 2000:

(1) Improving the rate of diffusion of sewerages:

By the year 2000, sewerages are to be fully arranged in urban areas not only for large cities but also for local cities and, in addition, sewerage arrangement required for improving living environment of rural districts and for conserving the water quality of lakes and swamps as good natural environments is to be carried out, thus raising the ratio between total population and sewerage-served population to those of developed countries in Europe and the Americas.

(2) Technically improving sewage treatment:

To achieve environmental quality standards for water and to prevent excessive nutrition in wide closed-type water bodies, measures to reduce pollutants at pollution sources are to be strengthened. At the same time, with respect to sewage works, tertiary treatment for removing organic matter and nutritious salts such as phosphorus is to be realized where it is required.

(3) Upgrading existing facilities:

To maintain the functions of publci sewerages, old defective facilities are to be remodeled and combined sewerages are to be upgraded depending upon the local conditions. Also for improvihg safety in urban areas, measures against floods are to be strengthened by raising the probability year related to the storm water removal plan in close coordination with flood control measures to be taken as part of river projects.

(4) Performing wide-scale treatment and disposal of sewage sludge:

To perform the proper treatment and disposal of sewage sludge, wide-scale treatment and disposal of sewage sludge is to be carried out in response to local conditions. Effective use of the sludge as, for example, organic fertilizer or as soil improvement materials is to be promoted by returning it to the natural cycle.

(5) Effecting recycled utilization of treated water from sewerage:

Treated water from sewerage is to be reconsidered as past of water resources and aggressively utilized. Particularly for areas where a short supply of water is expected, projects for reutilizing the treated water from sewerage are to be carried out.

(c) Immediate Arrangement Targets

In order to achieve the targets of the long-term arrangement of sewerage, the target of sewerage arrangement urgently needed is to raise the ratio of sewerage-served population to total population to 55% by 1985, based upon the fundamental concept of the 7-year New Economic Society Plan. The following points are to be emphasized during execution:

(1) Sewerage arrangement in local cities where the diffusion of sewer systems is presently slow is to be accelerated to promote the Integrated Residence Policy.

(2) Sewerage arrangement required for preventing floods in urban areas and for assuring higher safety in urban life is to be accelerated.

(3) Sewerage arrangement required to obtain the early fulfillment of the Public hazard prevention plan and of environmental quality standards for water established by the government and to respond to the total mass effluent control system in wide, closed-type water bodies is to be accelerated.

(4) To prevent excessive nutrition in wide, closed-type water bodies and lakes, tertiary treatment is to be accelerated in the water areas where great damage occurs due to red tides.

(5) Sewerages required for conserving the quality of water in tourist resorts and lakes with excellent natural scenery and sewerages for rural districts are to be encouraged.

(6) Wide-scale treatment and disposal of sewage sludge is to be accelerated. Effective utilization of sewage sludge and treated water from sewerages is to be accelerated.

5.1.2 Method of Sewerage Improvement and Arrangement

(a) Effective Promotion of Sewerage Arrangement

It has now been recognized that sewerages are required not only in large cities but also in small and intermediate local cities, and rural districts. Therefore, priority areas for promoting sewerage arrangement for the future have been shifted from large cities to local cities. Thus, sewerage projects will become more active in local cities and farming, forestry and fishing villages from now on. However, financial and technical powers in these local areas are generally smaller than those of large cities, so that it is considered to be more effective to promote wide-scale unified sewerage arrangement with centralized treatment plants provided for local cities and farming villages within the jurisdictions of at least two local government bodies, in view of investment efficiency, management efficiency, the administrating system for construction, maintenance and operation, and the overall effects on water quality conservation.

As methods for wide-scale sewerage arrangement, there are two methods; one of them is called the "wide-scale public sewerage method", in which public sewerage covering two or more cities, towns and/or villages is provided for and administered by a cooperative; the other method is called the "river basin sewerage method" in which planning and arrangement is made for each unit basin in two or more cities, towns and/or villages by the prefectural government as an administrator. It is required to fully utilize these methods in the future in order to promote the effective sewerage arrangement in local urban areas.

In addition, the population estimated for design purposes for these local areas is normally smaller than that of intermediate-scale cities, so that costs for constructing, maintaining and operating treatment plants per capita are higher, and also poorer efficiency in pipe installation will result since villages are scattered in these local areas.

When making sewerage arrangements for these small cities or villages, it is not necessarily rational to use conventional systems and treating methods from the viewpoint of promoting energy and resource conservation and of performing effective management and operation. Therefore, while promoting research and development work for energy saving techniques, it will be required to make efforts for the decentralization of treatment plants depending upon the degree of population concentration, for the positive adoption of easily maintainable, operable treatment facilities, for the adoption of economical treatment methods, for the simplification of treatment facilities, for the execution of cleaning and inspection of facilities, and for the execution of collection and monitoring of sewage sludge, whereby developing new forms of sewerage arrangement particularly suited to actual local conditions, instead of relying upon sewerage systems that have been developed mainly in large cities.

(b) Promotion of Tertiary Treatment

Through analysis on the amount of COD load by source in three water areas subjected to the total mass effluent control system and on the amount of generated phosphorus load in Seto Inland Sea, it was found that there is a large amount of generated load from domestic sewage in all cases. Thus, arrangement of sewerage has become more important for reducing the amount of generated load due to such domestic sewage.

Since COD and even phosphorus can be removed to a certain degree by secondary treatment, top priority must be given to the arrangement of sewerage in order to conserve water quality and to prevent excessive nutrition in water bodies subjected to total mass effluent control and lakes functioning as water resources, in view of the present situation of sewerage arrangement, which is progressing too slowly.

However, for preventing water pollution in these water bodies, it is of course required to strengthen overall measures against generating sources, such as the reduction of the amount of polluted load at the source, reduction of phosphorus in synthetic detergents and restricting their use, and the proper execution of culture-type fishery businesses. For conserving the quality of water in public water bodies, control of the generation of the polluted load is needed, instead of expecting too much sewerage, thus taking total conservation measures.

Based upon the above considerations, it may become necessary to organically introduce the tertiary treatment for water bodies where urgent measures are needed because of the considerable pollution and frequent occurrence of damage caused by red tide.

Though large construction costs will be needed for carrying out tertiary treatment, it must be understood that its benefit area is huge and not limited to that area served by sewarage. Conservation of the water environment as a national land resource is demanded from both the national and historical viewpoints, so that subsidies from the national government for tertiary treatment should be naturally different from that for the system up to secondary treatment.

Also, high costs will be paid for maintaining and operating the tertiary treatment facilities, but rules of allocation for the burdens have not been clearly defined as yet.

For solving this problem, it will be required to establish the proper allocation of burdens to be borne by the national government, prefectural government, city, town or village authority, and users depending upon local characteristics, in consideration of the range of each tertiary treatment plant and its purposes, such as the prevention of excessive nutrition in wide, closed-type water bodies, and the fulfillment of environmental quality standards for water in wide water conservation areas such as lakes. For example, with respect to the total development project of Lake Biwa, which is important as a precious drinking water source for the Kinki District, the allocation of burdens was determined after making an adjustment among the local public organs concerned. Beneficiary prefectures in downstream areas bear part of the expenses for the water resource development project. For tertiary treatment at lakes required for the conservation of wide-range water resources, the question of downstream areas bearing appropriate portions of the burden according to the benefits they receive may be worthy of examination.

(c) Promotion of Sewerage Arrangement as Flood Control Measures

Traditionally in Japan, inundation control is one of the important purposes of sewerage arrangement. Many cities in this country have frequent rains and located in slough areas. In addition, mountains are steep and storm water tends to run off very quickly.

In the 1960's, many large-scale housing land development projects were conducted in the outskirts of many cities in order to accept the population movement to large cities such as Tokyo and Osaka. Rapid development in these urban areas reduced paddy fields, green areas and vacant lots, further increased the storm water run-off coefficient as a result of the construction of buildings and the paving of roads, and immediately increased storm water run-off that was previously balanced, quickly increasing the possible danger of inundation in urban areas.

It is difficult to increase the widths of downstream rivers in response to increasing storm water run-off caused by the sudden development in upstream and middle-stream basins. Therefore, for these areas, execution of total flood control measures, including the arrangement of river facilities, control of storm water run-off by construction of regulating pondages and retention ponds, and the encouragement of safe land utilization will become necessary. Particularly for preventing flooding, all facilities related to storm water removal, including not only rivers but also storm water channels, must be provided in accordance with plans while maintaining the conformity of the overall facilities and the balance with each other.

(d) Harmony with Community

Construction of sewage treatment plants is frequently opposed by community residents because of their bad image with residents, the possible occurrence of odor and air pollution. This problem of opposition by residents frequently has occurred in this country because treatment plants have been constructed adjacent to residential areas or villages.

For this reason, local public bodies have tried to fully consider the possible impact to the nearby environment prior to the construction of sewage treatment plants, and have carried out environmental protection measures such as providing covers for treatment facilities, the installation of odor prevention facilities and air pollution prevention facilities, and planting within treatment plant sites in order to obtain the understanding and cooperation of community residents. The forceful execution of such environmental planning is also indispensable in the future. Particularly in view of the formation of a pleasant environment in the surrounding area, measures for such surrounding areas must be positively carried out; for example, sewage treatment plants should be constructed as part of parks associated with other urban park projects.

Especially for the treatment plants for river basin sewerage, answering people's opposition is more difficult than that of public sewage treatment plants since the former gives treatment not only for sewage from the local city or village, but also for sewage coming from other cities or villages.

Therefore, for an organic and integrated execution of environmental arrangement in the area near sewage treatment plants for the river basin sewerage, effective measures for the community must be taken very cautiously by the prefectural government as administrator for the river basin sewerage, in close collaboration with the relevant city, town or village as a beneficiary body.

(e) Promotion of Energy and Resource Conservation

Japan, where no oil is produced, takes the current world oil situation extremely seriously, and the government is trying to save energy and natural resources as much as possible in every field and propose that the whole economy of this country should shift toward energy and resource conservation systems.

Conventionally, the arrangement of sewerage in this country was mainly promoted in large cities, and efficient execution of the works was possible by adopting high-energy consumption type sewerage systems, such as activated sludge systems, but the sewage works came to a turning point in the severe world energy supply situation currently in progress.

In consideration of the diversified roles imposed on the sewerage and the high public function it offers, the sewerage treatment must perform energy and resource conservation and solve its own problems. Thus, from now on, it is very important to develop new technology for energy and resource conservation and to apply aggressively the results of research to actual sewerage.

Actually, for existing sewerage, it will be required to apply the energy-saving type operating and controlling method to such existing facilities and to encourage the switch to energy-saving type facilities. For new installations, it will be required to adopt the most effective energy-saving systems that are conceivable at present.

It is expected that sewerages will be widely adopted and constructed from now in small cities and rural districts, but efficiency of pipeline arrangement will become poor since the population is small in these areas. Construction cost for sewage treatment plant per capita will be relatively high when these villages are scattered.

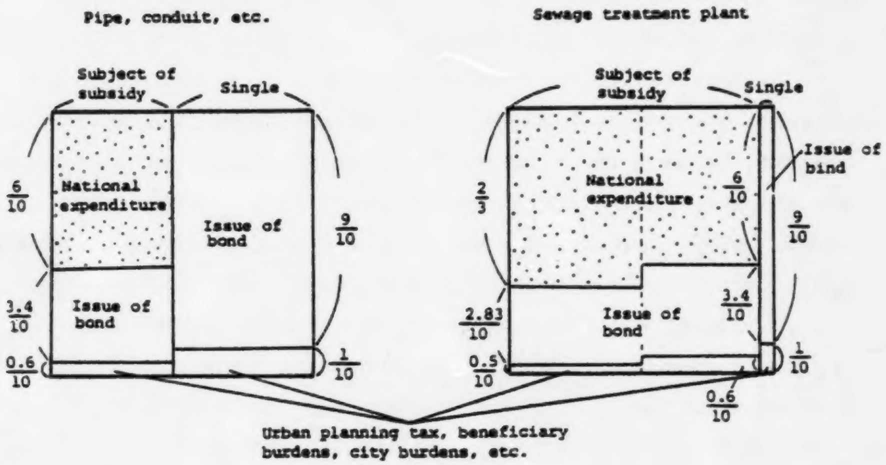
For promoting the sewerage arrangement in such areas, it is indispensable to adopt energy-saving type systems suited to the particular conditions of such areas.

5.1.3 Securing Financial Resources for Sewerage Arrangement

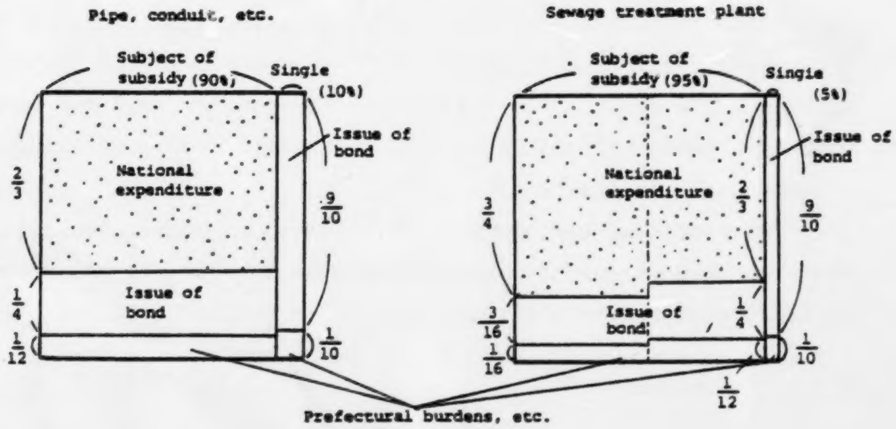
Current funding methods for sewerage arrangement slightly vary depending upon the types of sewerage system, but the funding methods for main public sewage works and river basin sewerages are as indicated in Fig. 3, that is, funds are provided in the form of national subsidies, local bonds, ordinary city burdens, prefectural burdens and beneficiary burdens.

Fig. 3

(1) Public sewerage


Pipe, conduit, etc.
Subject of subsidy
Single
National expenditure
Issue of bond
Issue of bond
6/10
3.4/10
0.6/10
9/10
1/10
Sewage treatment plant
Subject of subsidy
Single
Issue of bind
National expenditure
Issue of bond
2/3
6/10
9/10
2.83/10
3.4/10
0.5/10
1/10
0.6/10
Urban planning tax, beneficiary burdens, city burdens, etc.

(2) River basin sewerage


Pipe, conduit, etc.
Subject of subsidy (90%)
Single (10%)
Issue of bond
National expenditure
Issue of bond
2/3
9/10
1/4
1/12
1/10
Sewage treatment plant
Subject of subsidy (95%)
Single (5%)
Issue of bond
National expenditure
Issue of bond
3/4
2/3
9/10
3/16
1/4
1/16
1/10
1/12
Prefectural burdens, etc.

For the last several years, financial resources for each sewage work has steadily expanded, and particularly natioaal subsidies and local bonds have grown considerably. It may be said that sewage works have been expanded mainly because of the expansion of these subsidies and bonds.

On the other hand, under present circumstances where the national economy of Jpaan is forced to shift to stable growth, it can be easily forecasted that both national and local governments will have more tight and inflexible situations financially. This situation must be fully taken into account in carrying out the sewage construction.

Particularly for small and intermediate local cities where sewerage arrangement is specially needed, it will be required to create a new system for securing financial resources which is different from the present one in consideration of their current financial conditions, in order to aggressively promote sewerage arrangement for the future.

5.2 Maintenance and Management of Sewerage

5.2.1 Method of Maintenance and Management

The final objective of sewage construction is not to construct the facilities but to properly operate and control them.

In consideration of the highly public character of the system, each sewerage administrator is required to make every effort for the optimum maintenance and operation of facilities.

By reviewing the problems associated with sewerage maintenance and oepration in this country, the following items are considered to be the most important problems that are to be solved from now.

(a) Wide-scale Disposal of Sewage Sludge

It is expected that the amount of sewage sludge from sewage treatment plants will increase considerably as sewage works progress in the future, but work will be finally completed only after sewage sludge is properly treated and disposed. For stably treating and disposing the sludge, its effective use has to be positively promoted and reclamation land for the disposal of sludge must be aggressively secured.

On the other hand, securing disposal and reclamation lands has become very difficult as a result of progress in urbanization. In addition, under current circumstances, where incineration cannot be easily performed because of the increasing demand for energy conservation, it can be said that the disposal of sewage sludge is now creating very serious problems.

For stably disposing the large amount of sludge expected in the future, it will be more effective to make wide-scale plan, for treating and disposing sewage sludge for a wider district, not limited to the relevant prefecture, city, town, or village and to treat and dispose of the sewage sludge by each sewerage administrator in conformity to this plan.

Particularly in large urban zones such as Tokyo and Osaka where sewerage arrangement must be quickly promoted in response to the total quantity control in the future and tertiary treatment has to be primarily conducted as required, the generation of a huge amount of sludge is forecasted. But securing sludge disposal land is now very difficult, which makes disposal within the city area almost impossible for cities located in these large urban zones.

Thus, in large urban zones where securing sludge disposal lands as a prerequisite for sewerage arrangement is very difficult, it is very hard to use conventional methods in which the individual sewerage administrator can dispose of it independently. As a result, there is a tendency for a prefectural or national government body to act as an executing organ for arranging fundamental conditions for the sludge treatment and disposal, based upon the wide-scale approach through coordination not limited to the single prefecture, city, town or village.

(b) Strengthening of Control over Pre-treatment Facilities

To achieve the purpose of sewerage arrangement and to effectively perform sewerage arrangement, the sewerage law will impose "compulsory use" that is unusual for public property laws, so that the sewerage must accept as a rule all domestic and industrial wastewater, regardless of its origins.

When accepting the industrial wastewater, it is requested that a factory generating sewage harmful to functions of sewage facilities and materials that cannot be removed in sewerage, must perform pretreatment for such sewage within the factory in advance to reduce such harmful matters below the minimum standard level. Thus, such a factory is legally obligated to install pre-treatment facilities.

Particularly for harmful materials such as heavy metals, quality standard similar to that of sewage discharged to ordinary public water bodies has been determined. Thus, it is said that an important water quality conservation policy comparable to that of pollution control in public water bodies is being effected through the sewerages.

With respect to the sewerage law, provisions for control of water quality were prepared in 1976, in which, to cope with harmful sewage, a new direct penalty system, advance check system and correction order system were introduced in addition to the conventional system for compulsory installation of pre-treatment facilities.

The number of factories and work places where the installation of pre-treatment facilities is obligated within sewerage treatment zones was about 20,000 at the end of 1979, and more than 15,000 factories installed such required facilities. This is considered to be an improvement since the rate of installation of such facilities at the end of 1979 is about 85% compared to 63% at the end of 1975.

However, there are still about 3,000 factories that have not installed such pre-treatment facilities.

Most factories which have not installed the required facilities are small establishments, and more than 70% of them indicated that they could not install such facilities because of funding problems and the difficulty of finding proper land for the facilities.

For the purpose of ensuring the pre-treatment activities by such small estbalishments, it is required to positively promote joint-pretreatment through the preparation of industrial zones for small firms, measures for ensuring long-term low-interest loans for the pre-treatment facilities, and offering necessary guidance and technical assistances by sewerage administrators, as part of the policies for the small business.

At the same time, the public sewerage sector should strengthen its monitoring and patrol system including the arrangement of monitoring instruments and the securing of monitoring personnel, while maintaining tight coordination with environmental administration.

(c) Promotion of Effective Use of Resources

While maintaining the optimum maintenance and control for the sewerage, the problems to be coped with in the future will be the promotion of the effective utilization of products made from the sewerage, that is, treated water and sewage sludge.

Effective utilization of these resources will also, in consequence, solve problems in sewerage maintenance and operation, and this is considered to be an extremely effective measure, immediately responding to the limited resource age in this country.

(1) Effective Utilization of Sewage Sludge

As already stated before, it is important to consider that sewage sludge is to be effectively utilized as a resource as much as possible instead of merely disposing of it, for the purpose of stabilizing sludge disposal. At present, only 10% of total sludge is effectively being used, mainly as fertilizer or soil improvement material by returning it to agricultural lands and green areas.

It is important to promote this use for agricultural lands and green areas also in the future. For this purpose, it is indispensable to make the sludge easier to handle by users and to obtain the trust of users. For this purpose, a system related to sludge quality control and sludge distribution must be established.

While promoting the use of sewage sludge in agricultural lands and green areas, research and development work for utilizing the sewage sludge as raw material for construction materials such as concrete aggregates, bricks and concrete blocks and for recovering valuable materials such as phosphorous and aluminum from the sewage sludge, must be aggressively carried out.

(2) Reutilization of Effluent from Sewerage

Water resource development in this country has been conducted so extensively that it seems to be very difficult to secure new water resources. Particularly in large cities, the water supply-demand relationship is extremely unstable as a result of the excessive concentration of population and industries.

In order to protect sound social activities in these areas in the future, reutilization of effluent from sewerage must be considered in addition to more rational use of water by saving and other means.

Reutilization of effluent from sewerage is considered as water for miscellaneous urban use such as water for toilets and cleaning water, for industrial water, for agricultural water, and for gardening.

In Japan, effluent from sewerage is already being used on a small scale but, except for a few examples, the real application of effluent has not yet been started.

Reutilization of effluent from sewerage will be considerably increased in the future and, for its actual application, it will become necessary to clearly define the position of effluent as a water resource and to establish an overall plan for water demand and supply. In addition, various surveys establishing water quality standards and facility standard suited to each application must be quickly made to promote the efficient reutilization of the effluent.

(d) Arrangement of Executing System

Sewerage facilities must be properly operated at all time and their usefulness will be realized only when they are properly maintained and controlled. Discontinuity in services after the commencement of treatment by these facilities is not permitted. Therefore, the arrangement of a complete system is indispensable during planning and operation for these facilities. Thus, provisions of Article 12 of the sewerage law require that engineers engaging in planning, designing, supervising, maintaining and operating the sewerage plants are to have predetermined qualifications. However, at present, qualified engineers are not necessarily sufficient in ordinary cities and become insufficient as the size of the city gets smaller.

For steadily promoting sewage construction in small and intermediate cities in the future, it is urgently required to secure engineers who engage in construction work. At the same time, securing maintenance and operating engineers is also urgently needed in order to perform maintenance and operation work appropriately, such as control of effluent, treatment and disposal of sludge, and monitoring of inferior sewage in response to the commencement of sewerage treatment. For this purpose, it is required not only to secure personnel and provide organization in relation to the sewage works, but also to improve the capabilities of each engineer through effective training. In arranging the executing system under circumstances, where demand for simplified administrative organization is increasing, it is required to strengthen cooperation with other departments such as those in the environmental sector, to cooperate with adjacent regional public bodies, and to promote mutual cooperation in joint-pre-treatment and technical assistance with such bodies.

In consideration of the role of sewerage in water quality conservation in public water bodies, basic work such as water quality monitoring should be directly controlled by the sewerage administrator. Work such as simple labor should be contracted to outside private firms in order to effectively achieve the work load within the limited administrative syst

5.2.2 Securing Financial Sources for Maintenance and Management

Conventionally, maintenance and operating costs for the sewerage treatment were provided by the "private burden for sewer" and "public burden for storm water" and, as a rule, the portion related to sewage was borne by users of sewerage in the form of a sewerage service charge. The portion related to storm water was borne by the city since the latter was related to a natural phenomenon and thus charging citizens was not considered practical.

The number of cities, towns and villages that begin the joint-use of sewerage will continuously increase in the future, and securing financial sources for proper maintenance and operation will become indispensable for effectively achieving the role of sewerage treatment.

In order to secure financial sources for the maintenance and operation of the sewerage plants, the public role related to sewerage services must be taken into account, and it will be required to establish a rule by which proper burdens will be imposed to users, in addition to enlarging financial measures for the burden to be borne by public bodies.

Users of sewerage services can be known and clearly determined but, on the other hand, they are also obligated to pay the appropriate social cost as originators of water pollution, so that it is considered that the principle of the "private burden for sewer" and "public burden for storm water" should be also be applied in the future in securing financial sources for service maintenance and operation.

Seventh US/Japan Conference
on
Sewage Treatment Technology

CURRENT ISSUES IN WATER POLLUTION CONTROL ADMINISTRATION IN JAPAN

May 19, 1980
Tokyo, Japan

Toshiki Oshio
Environment Agency
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1. CURRENT STATE OF POLLUTION IN JAPANESE WATERS

Although still very far from a satisfactory state, an overall improvement of water quality may be noted in Japanese waters in recent years. Let me start by examining the pollutants for which environmental water quality standards have been established under the provisions of the Basic Law for Environmental Pollution Control. As regards toxic or harmful substances relating to protection of human health such as cadmium, mercury and PCBs, the ratio of samples exceeding the respective standards to the total number of samples taken has continued to decline over the years. (See Table 1.)

Table 1. Ratio of Samples Exceeding Water Quality Standards Relating to Protection of Human Health To The Total Number of Samples Taken

Substances	Water Quality Standards	1970 (%)	1975 (%)	1978 (%)
Cadmium	0.01 ppm	2.8	0.31	0.15
Cyanide	N.D.	1.5	0.02	0.01
Organic P	N.D.	0.2	0	0
Lead	0.1 ppm	2.7	0.32	0.05
Cromium (VI)	0.05 ppm	0.8	0.02	0.01
Arsenic	0.05 ppm	1.0	0.24	0.15
Total Mercury	0.0005 ppm*	1.0	0**	0**
Alkylmercury	N.D.	0	0	0
PCBs	N.D.	–	0.38	0.02

(* Annual averages; ** In number of sampling sites)

Among a number of substances and items for which there exist water quality standards relating to the preservation of our living environment, taking BOD levels in rivers for example (See Table 2), a general trend toward improvement may again be noted here. The non-compliance rate, however, is high among small rivers and streams flowing through large urban districts. The rate of achievement of environmental standards in 1978 was: 61% for water bodies belonging to Category AA, 71% for Category A, 52% for

Category B, 42% for Category C, 48% for Category D, and 40% for Category E, with an overall achievement rate of 60%.

Table 2. Ratio of Samples Exceeding Water Quality Standard for BOD in Rivers

Water Quality Categories	Water Quality Standard for BOD	1971	1975	1978	(Number of Water Bodies)
AA	1 ppm	36.7	31.4	28.1	(275)
A	2 "	30.9	24.4	23.6	(980)
B	3 "	35.6	27.4	31.9	(498)
C	5 "	39.9	42.6	45.0	(225)
D	8 "	52.8	37.8	30.6	(80)
E	10 "	70.2	49.7	46.2	(139)

Taking for another example the number of coliform groups, one would get a non-compliance rate of 79% for all water bodies in Category AA (the standard is less than 50 MPN/100 ml), 70% for Category A (ditto, less than 1,000 MPN/100 ml), and 56% for Category B (ditto, less than 5,000 MPN/100 ml). No improvement can be seen in this respect.

Turning next to COD levels in coastal waters, a general trend for improvement may again be recognized, but the rate of achievement in 1978 was: 59% of water bodies in Category A, 79% in Category B, 99% in Category C, with an overall achievement rate of 75%. The rate was particularly low in the "enclosed" or "semi-enclosed" bodies of water. (See Table 3)

Table 3. Ratio of Samples Exceeding Water Quality Standard for COD in Coastal Waters to the Total Number of Samples Taken

Water Quality Categories	Water Quality Standard for COD	1971	1975	1978	(Number of Water Bodies)
A	2 ppm	38.5	19.6	17.6	(215)
B	3 "	30.5	18.3	17.1	(190)
C	8 "	15.2	7.4	6.1	(117)

Looking at COD levels in lakes and reservoirs, here again one may note an overall trend for improvement, but the actual rate of achievement of water quality standard is extremely low, as illustrated by the achievement rates of 29% of water bodies in Cagetory AA, 47% in Category A, only 14% in Category B, with an overall average rate of 38%. (See Table 4)

Table 4. Rate of Achievement of the COD Standard for Lakes and Reservoirs

Water Quality Categories	Water Quality Standard for COD	1971	1975	1978	(Number of Water Bodies)
AA	1 ppm	13.9	62.0	66.9	(21)
A	2 "	79.4	69.4	63.3	(57)
B	5 "	91.8	84.6	73.0	(14)
C	8 "	–	40.7	41.7	(1)

2. WATER POLLUTION IN SEMI-ENCLOSED COASTAL WATERS AND AN AREAWIDE POLLUTION LOAD CONTROL SYSTEM

In the three metropolitan regions of Tokyo Bay, Ise Bay and Seto Inland Sea lives 53% of the total population of Japan, and 65% of manufactured goods are produced by these regions. As naturally to be expected, a large amount of pollutants is discharged into these so-called "enclosed" water areas. The flushing rate, or turnover of water with the outer fringes of the ocean, is very small, and organic substances are easily retained and accumulate in these water bodies. Due to the great influx of nutrients, moreover, the process of eutrophication is in rapid progress. As a result, the water pollution problem for these waters is getting worse and more complex every year.

In order to cope with this worsening progress of water pollution, the 84th session of the National Diet in 1978 enacted a Special Measures Law for Seto Inland Sea Environmental Preservation and amendments to the Water Pollution Control Law, both of which went into effect in June, 1979. With the objective of achieving and maintaining environmental water quality standards, the new Law and the amendments of 1978 introduced a system of measures aimed at an "areawide control of water pollution load", which sought to cut down on the amount of total pollutant loadings into the three water areas in an effective and integrated manner, from all sources including domestic households and hydraulic load of rivers in upstream inland areas.

Targeted reduction in COD load, according to the type of sources and to each prefecture, goals for raising the percentage of sewered population, basic guidelines on setting new effluent control standares, etc. were to be specified in a comprehensive Pollutant Reduction Plan. The Plan was approved by the Prime Minister in March, 1980, which set a deadline of 1984 for reducing the COD loadings to the levels shown in Table 5.

Table 5. General Features of Tokyo Bay, Ise Bay and Seto Inland Sea regions and the Goals for Reduction of COD Load

	Tokyo Bay	Ise Bay	Seto Inland Sea
Surface Area of Water Body (km^2)	1,400	2,300	23,000
Volume of Water (×100 million m^3)	540	460	7,330
Population (1976) (×1,000)	22,200	9,090	28,130
Manufactured Goods (1976) (trillion yen)	32.2	18.4	44.5
COD Loadings in 1984 (target) (tons/day)	660	426	1,283
Domestic Sources	386	179	517
Industrial Sources	180	208	666
Others	94	39	100

In this connection it is important to note that expanding sewerage networks is an essential requirement for the achievement of these goals, as indicated by the still very low rate of sewered population in these regions: in 1977, it was 38% in the Tokyo Bay region, 27% in the Ise Bay region, and 31% in the Seto Inland Sea region.

Along with reduction of COD loadings, inflow of nutrients must be controlled so as to reduce production of organic material by phyto-planktons utilizing those nutrients. In 1977, for example, 24% (680,000 tons) of all fishery products in Japanese coastal waters was raised in the Seto Inland Sea. The number of "red tides" observed in the Sea, which was 48 in 1967, jumped from 79 in 1970 to 320 in 1975 and 165 in 1978. The red tides along with them brought heavy damages to fishery production in the Seto Inland Sea, and became an important issue to be urgently tackled with.

In the face of this event, measures were taken under the Provisional Law for Seto Inland Sea Environmental Preservation to reduce by 1976 the COD loadings from industrial sources by more than one half of 1972 levels. Thus, the average COD concentration in the whole Sea area in 1977 was brought to 1.7 ppm, while its transparency (Secci depth) was 5.8 meters, Total N 0.28 ppm, Total P 0.029 ppm. The overall rate of achievement of water quality standards was 75% (40% for Category A, 80% for Category B, and 98% for Category C). Compared with national averages, the achievement rate of water bodies in Category A, which make up a major part of the Inland Sea, is very low.

Furthermore, daily inflow of phosphorous into the Seto Inland Sea reaches as high as 81 tons per day. (34 tons from domestic sources, 33 tons from inductrial sources, and 14 tons form other sources).

In order to raise the water quality of the Seto Inland Sea, the Special Measures Law for Seto Inland Sea Environmental Preservation provides for measures to reduce the input of nutrients into the Sea area. On the basis of this Law, Director-General of the Environment Agency in July, 1979 directed the governors of relevant prefectures to draw up plans, including specific measures, to reduce or maintain the present level of P loadings into the Sea by 1984, depending on the prevailing state of pollution by the nutrient. Further studies will be conducted regarding possibilities for reduction of P concentrations in Tokyo and Ise Bays.

3. EUTROPHICATION OF LAKES AND RESERVOIRS

Of the 93 lakes and reservoirs for which water quality data are available, 54 show a pH of 8.5 or above, and a DO of more than 10 ppm. In these lakes, pH rises as CO_2 is taken up by the multiplying population of algae, and DO increases to the extent it becomes super-saturated. Many of the lakes and reservoirs have thus become enriched with nutrients. Lake Suwa, for example, had a nitrogen concentration of 0.26 ppm and phosphorous concentration of 0.02 ppm in 1931, whereas the corresponding figures rose to 3.51 ppm and 2.47 ppm respectively in 1978.

Table 6. COD Levels in The Major Lakes of Japan

Name of Lake	Water Quality Category	Maximum Concentrations Observed	Daily Means at Standard Sampling Sites (75% Values)	75%V/ W.Q.S. Values
Tega-numa	B	49 ppm	28 ppm	5.6
Mikata-goko	B	72 "	22 "	4.4
L. Kitagata	B	16 "	15 "	3.0
L. Suwa	A	164 "	15 "	5.0
Kasumiga-ura	A	112 "	14 "	4.7
Hachiro-gata	A	26 "	14 "	4.7
L. Biwa	AA	7.5 "	5.7 "	5.7

It should also be noted that Japanese lakes, many of which are still in an oligotrophic state and have long been associated with one of the greatest transparencies in the world, are rapidly deteriorating in terms of their clarity/ transparency. Secci depth of Lake Shikotsu, for example, decreased from 25 meters in 1925 to 18.7 meters in 1978, while in Lake Towada it was reduced from 20.5 meters in 1930 to a mere 9.8 meters in 1978. The phenomenon of lake eutrophication is not restricted to natural lakes, either. It now extends to many of the man-made reservoirs constructed for drinking water purposes. Of a total of 155 revervoirs surveyed in 1979, 55 is reported to have suffered cases of fungus-like odor and taste in the water. In other cases, filters were clogged by mass populations of algae and filtering efficiency lowered.

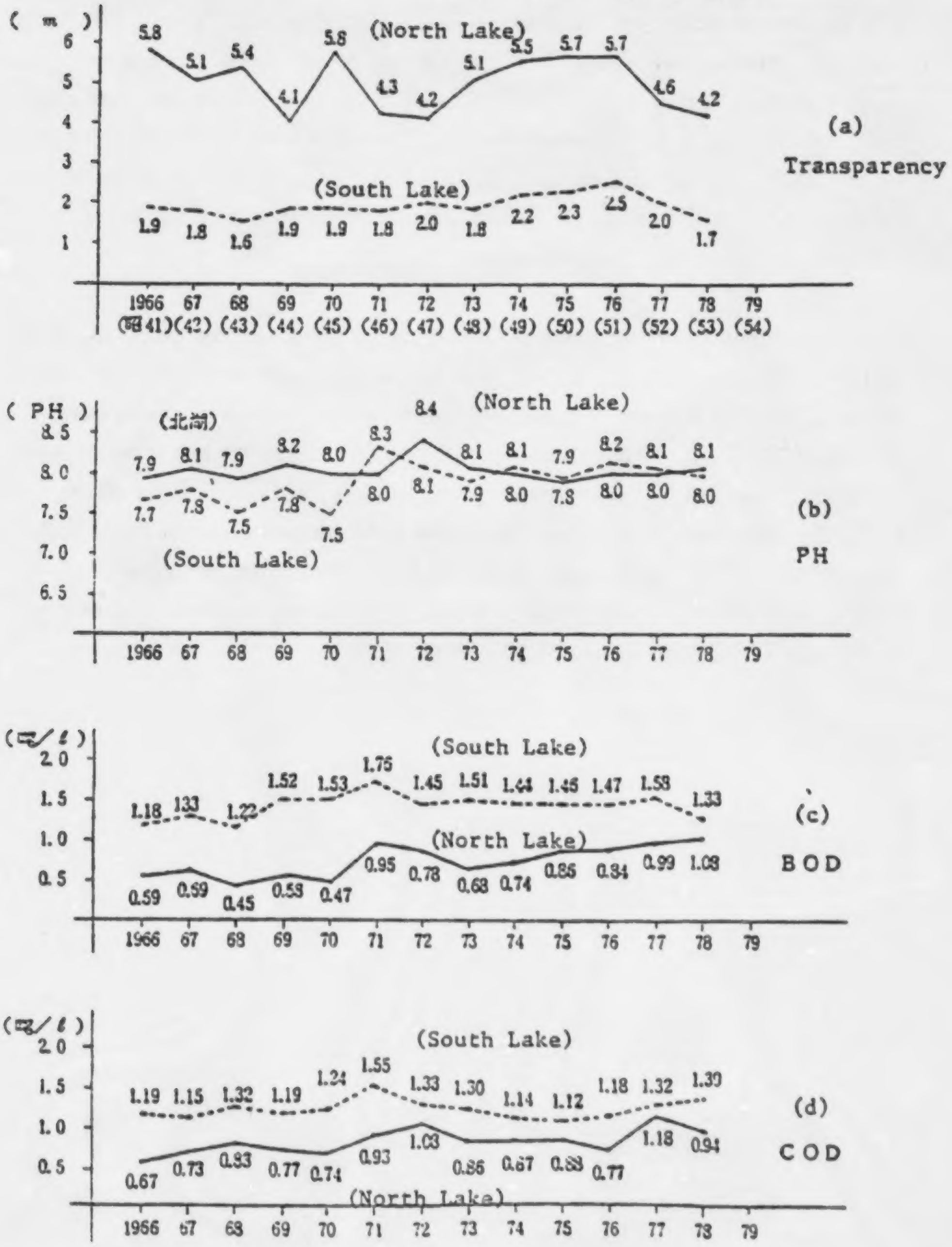
4. A CASE STUDY OF LAKE BIWA AND EFFORTS TO CONTROL ITS EUTROPHICATION

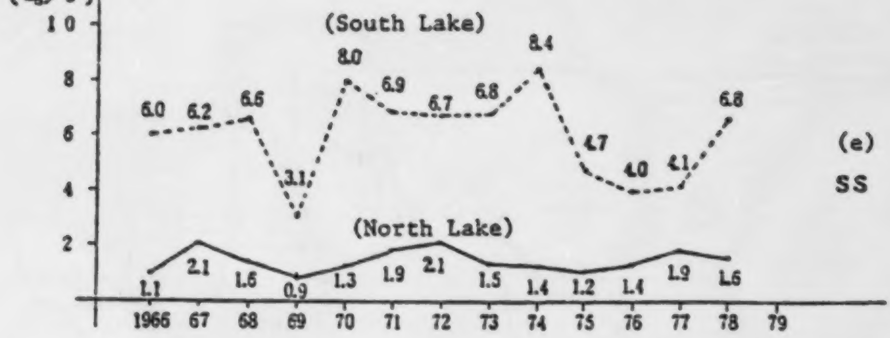
Lake Biwa is unique among Japan's more than 500 freshwater lakes. It is by far the largest of them all, with a surface area of 673 km^2, maximum depth of 103.6 meters, and 27.5 billion tons of water stored. Yet it is situated entirely within a single prefecture of Shiga, a relatively less developed region of this highly industrialized nation. The Lake supplies a large volume of water (4,000 to 6,000 million tons annually) to the metropolitan region of Osaka and Kyoto with over 13 million population as their sole source of industrial, irrigation and drinking water.

Topographically, the Lake is divided into two distinctive parts, the small southern end called the South Lake and by far the larger one the North Lake. As far as the North Lake is concerned, water quality indices of transparency, pH, SS, BOD, COD, Total N and Total P remained more or less constant over the years. See Figure 1. In recent years, however, and particularly since 1977, the "red tides" which had heretofore been restricted to the South Lake alone spread to the North Lake, which may by all accounts be called "oligotrophic" in the present state. Figure 2 marks specific areas and extent of red tide observation during May – June 1979.

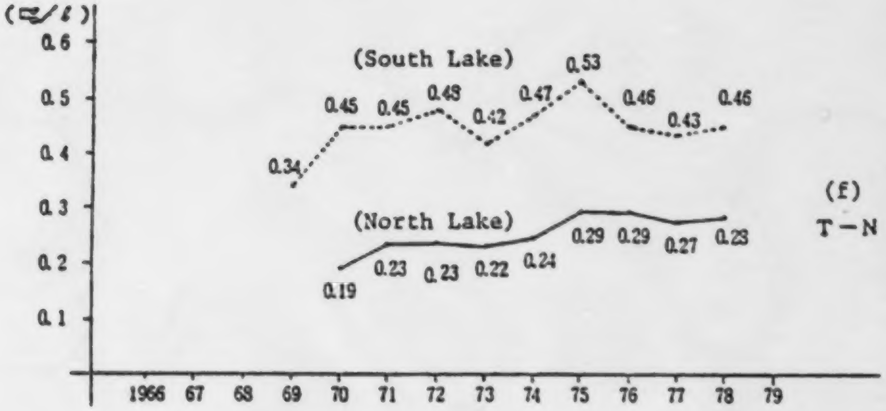
Figure 1. Water Quality in Lake Biwa

(annual mean of daily averages
at 48 monitoring stations)

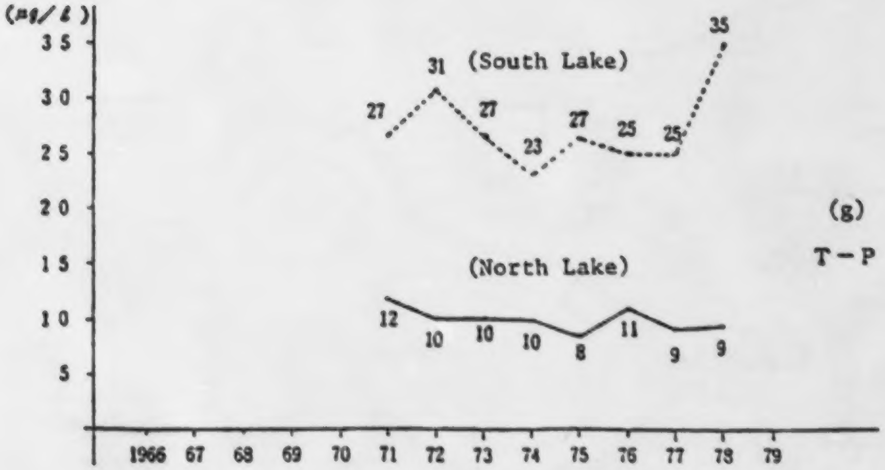




(e)
SS



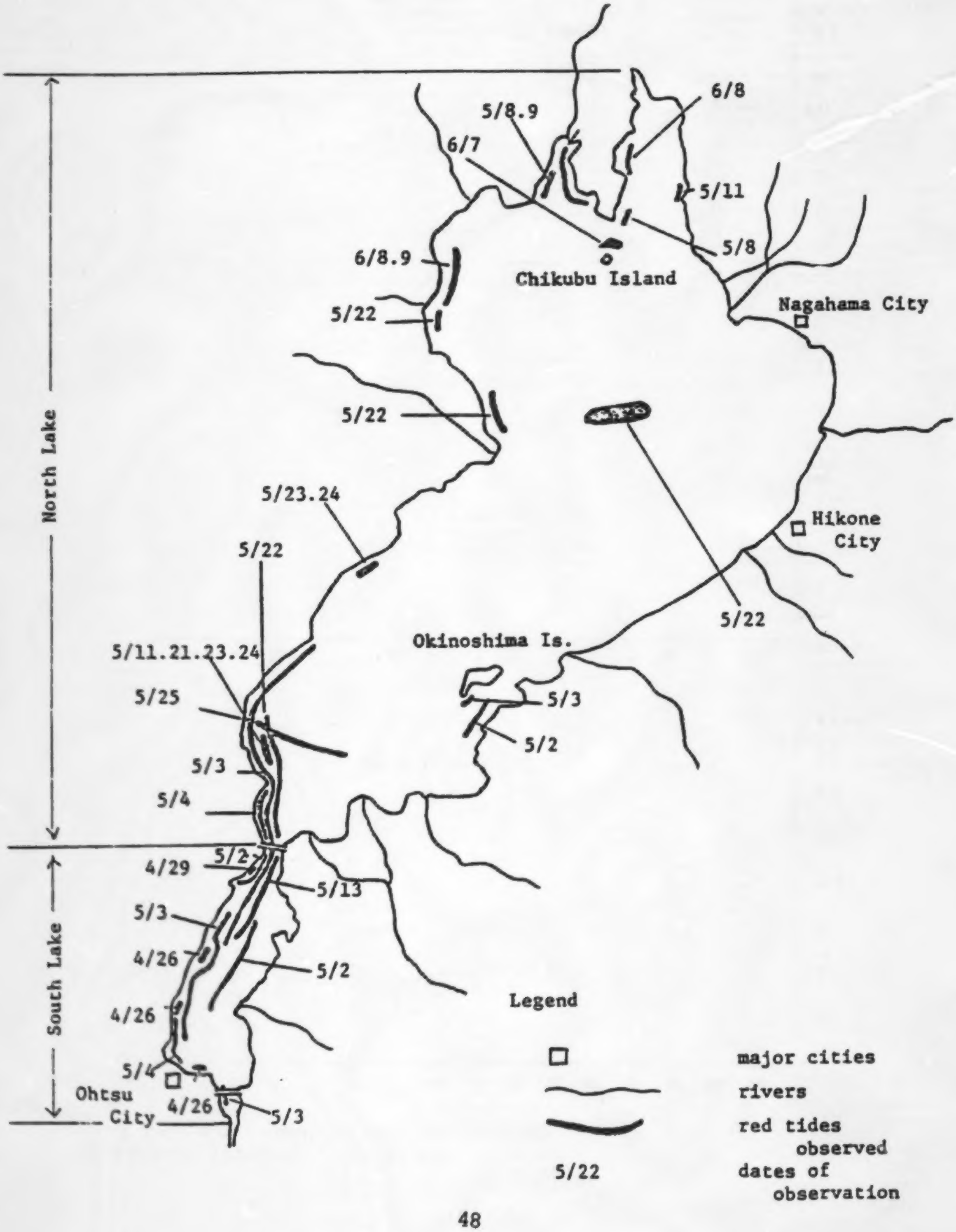
(f)
T－N



(g)
T－P

(Source: Environment Agency and
the Shiga Prefectural Government)

Fig. 2. Algal Blooms Sighted on Lake Biwa (1979)


6/8
5/8.9
6/7
5/11
5/8
6/8.9
Chikubu Island
Nagahama City
5/22
5/22
North Lake
5/23.24
Hikone City
5/22
5/22
Okinoshima Is.
5/11.21.23.24
5/25
5/3
5/2
5/3
5/4
5/2
4/29
5/13
South Lake
5/3
4/26
5/2
4/26
Legend
5/4
Ohtsu City
4/26
5/3
major cities
rivers
red tides observed
5/22
dates of observation

Clearly, phosphorus is not the sole cause of eutrophication, but recognizing that it is a key element in our efforts to prevent further deterioration of the nation's waters, the Government of Japan, though not backed up by any law, has in the past "guided" the detergent manufacturers to reduce the phosphate content in their products. The industry responded with a "voluntary restraint" in the use of phosphates, usually in the form of sodium tripoly-phosphate or $Na_5P_3O_{10}$, as a "builder" in detergents. Thus, the phosphate limit in detergents, which was as high as 20% by weight (in P_2O_5 equivalent) before, was reduced to 15% beginning in January 1975, to 12% as from January 1976, and further down to 10% in January 1979. Owing to the "administrative guidance" of the Ministry of Trade and Industry and to the detergent industry's own "voluntary restrictions", the P_2O_5 content in powdered detergents for household use has continued to decline over the years, although total production of phosphate-detergents gradually recovered from an all-time low of 409,000 tons in 1975 (peak production of 624,000 tons in 1974). See Table 7.

According to an estimade made by the prefectural government, 48% of phosphorus loadings into Lake Biwa is generated by domestic sources (general households), of which 38%, or 18.2% of the total input, is caused by phosphates contained in detergents for household use. See Talbe 8. The percentage of sewered population is extremely low, a mere 5% in Shiga Prefecture.

Table 7. Annual Production in Japan of Detergents and Soap

(Unit: 1,000 tons)

	1975	1976	1977	1978
A. Synthetic detergents				
1. for household use	592	679	719	823
a. powder	409	450	474	532
b. liquid	183	229	245	291
(1) for laundry				45
(2) for dish-washing				192
(3) for other household uses				54
2. for industrial use	269	301	328	365
B. Soap	122	147	151	156
1. for laundry				
a. powder	21	23	25	27
b. solid	11	11	12	11
2. for bath use	75	96	98	104
3. for industrial use	15	17	16	15

Source: Statistical Yearbook of Chemical Industries)

Table 8. Phosphorus Loadings into Major Lakes of Japan by Source

Sources	Lake Biwa (1975)		Lake Kasumigaura (1973)		Lake Suwa (1973)	
	(kg/day)	(%)	(kg/day)	(%)	(kg/day)	(%)
Domestic	1,117	48.0	409	31.0	184	48.0
Factories	682	29.3	349	27.0	42	11.0
Animal feedlots	325	13.9	330	25.0	20	5.0
Agriculture			218	17.0	80	21.0
Forestry and others	205	8.8	–	–	55	15.0
(Total)	2,329	100.0	1,306	100.0	381	100.0

(Breakdown of domestic sources
for Lake Biwa only)

Night soil	18.7% of total
Detergents	18.2%
Direct drainage into the Lake and rivers	11.1%
	48.0%

(Source: for Lake Biwa . . . . . Shiga Prefectural Government
for Lake Kasumigaura and Lake Suwa . . . . . Environment Agency)

On October 16th, 1979 when its legislature unanimously passed a bill entitled the "Prefectural Ordinance for the Control of Eutrophication of Lake Biwa", Shiga Prefecture became the first in Japan to not only totally ban the sale and use of phosphate-containing detergents but also regulate nitrogen and phosphorus levels in effluents from industrial plants and public sewage treatment facilities. The Ordinance, which was promulgated the next day to take effect within one year (presently scheduled to go into effect as of July 1, 1980), prohibits any act of selling, displaying for sale or supplying household detergents containing phosphates and strictly forbids any person to use them or present them to others as a gift. The Ordinance further empowers the prefectural government to establish effluent discharge standards (maximum permissible levels) for nitrogen and phosphorus discharged from factories and other industrial facilities, sewage treatment plants and private septic tanks above a certain size.

By taking this bold step forward, the government and people of Shiga demonstrated their will and determination to protect, even at their own expense, the nation's single largest source of clean water from the threat of eutrophication, and thus set an important precedent for the other prefectures. The Ordinance at the same time authorized the prefectural government to regulate factory effluents in terms of phosphorus and nitrogen concentrations. Specific values for the effluent standards are to be set and promulgated in a separate regulation pursuant to the provisions of the new Ordinance.

It would be the basic objective of the new effluent regulations to bring down the level of phosphorus concentration in the lake water by 1985 to that at least of the period 1965-70, and to maintain, or at least not to deteriorate from, the nitrogen level existing in 1975. In more concrete terms, the targeted levels of water quality for nitrogen and phosphorus are shown in Table 9, which also provides a comparison between the present state and the future expected state with and without enforcement of the proposed new regulations in terms of nitrogen and phosphorus concentrations as well as in absolute amounts of inputs of the two nutrients into Lake Biwa. An estimated 111 factories or about 20% of all existing industrial plants and facilities will be required to install additional or new pollution control equipment in order to meet the stiff regulatory standards.

In addition, the new Ordinance instructs the governor of Shiga to take appropriate measures to reduce nutrient inputs into the Lake as a whole, including provision of technical advice and guidance to the farmers, dairy farms, forestry operators and households on proper management techniques and practices. In other words, it is the aim of the Ordinance to enable the prefectural government to take a comprehensive approach in combatting man-caused eutrophication of Lake Biwa, from point as well as non-point sourc sources. The Ordinance taken together with proposed new regulations, therefore, may be regarded as a form of "area-wide total pollution load control system" adopted by and applicable to a single Prefecture of Shiga.

Table 9. Water Quality Targets for Lake Biwa in Relation to Proposed Eutrophication Control Measures

		1975		1985 without control		1985 targets with control	
		Nitrogen	Phosphorus	Nitrogen	Phosphorus	Nitrogen	Phosphorus
Water Quality	North Lake	0.28 mg/l	0.010 mg/l	0.31 mg/l	0.017 mg/l	0.25 mg/l (estimates) 0.25-26	0.010 mg/l 0.009-11
	South Lake	0.33	0.015	0.40	0.023	0.30 (estimates) 0.32-33	0.015 0.012-14
Total Loading at Sources		21,454 kg/day	2,329 kg/day	28,656 kg/day	3,539 kg/day	21,982 kg/day (rate of reduction) 23 %	1,876 kg/day 47 %
Framework for estimation (Assumptions)		Population: 985,621 Production: 1,526.5 billion yen		Population: 1,230,000 Production: 3,267.5 billion yen (at 1975 prices)			

(Source: Shiga Prefectural Government based on a simulation model)

5. INTEGRATED APPROACH TO EUTROPHICATION CONTROL

In order to facilitate adoption and implementation of comprehensive measures to control eutrophication, Environment Agency decided to establish a set of "water quality objectives" for phosphorous and nitrogen, and in December 1979 convened a group of experts to consider and recommend appropriate levels of water quality regarding nitrogen and phosphorous with a view to controlling, and wherever possible preventing, the progress of eutrophication in the nation's various enclosed or semi-enclosed waters. The group is expected to finalize its recommendations by the summer of 1980 as far as the water quality objective for phosphorous levels in lakes and reservoirs is concerned. The objectives for P concentrations in other areas and those for nitrogen are expected to take much longer to materialize.

In March 1980 the Environment Agency also announced its plans for developing an integrated approach to eutrophication control, which included, among other things, an appeal to all government ministries and agencies to refrain from using phosphate-containing detergents at their organizations and affiliated institutions, including public schools and government-run hospitals and clinics. Notable among other measures to be taken were: accelerated deployment of sewerage networks, ensuring appropriate maintenance of individual household septic tanks, management of farming practices, mitigation and recycling use of wastes from dairy industry, further research into methods of control of non-point sources, development of technology for advanced waste water treatment, etc.

Seventh US/Japan Conference
on
Sewage Treatment Technology

REGIONAL WASTEWATER TREATMENT PLANNING IN TOKYO BAY AREA

May 19, 1980
Tokyo, Japan

Tsutomu Tamaki
Head,
Sewage Works Division,
Department of Sewerage and Sewage Purification,
Ministry of Construction

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Regional Wastewater Treatment Planning in Tokyo Bay Area

Contents

1. Outline of the Area 59

1.1 Natural Conditions 59

1.2 Social Conditions 61

2. Water Quality 64

2.1 Present Condition of Water Quality 64

2.2 Influent Pollution Load 67

2.3 Water Quality Simulation Models 72

3. Environmental Standards and Measures for their Achievement 76

3.1 Environmental Standards and Current Water Quality 76

3.2 Allowable Load 76

4. Present Status and Problems of Sewerage Development and Sewage Sludge Treatment and Disposal in Tokyo Bay Area 79

5. Outline of Regional Sewerage Plan 83

5.1 Advanced Wastewater Treatment (AWT) Plan 83

5.2 Sludge Treatment and Disposal Plan 83

5.3 Outline of Regional Sewage Sludge Treatment and Disposal Plan 85

5.4 Effects of Regional Sewage Plan 86

5.5 Future Problems 88

REGIONAL WASTEWATER TREATMENT PLANNING IN TOKYO BAY AREA

1. OUTLINE OF THE AREA

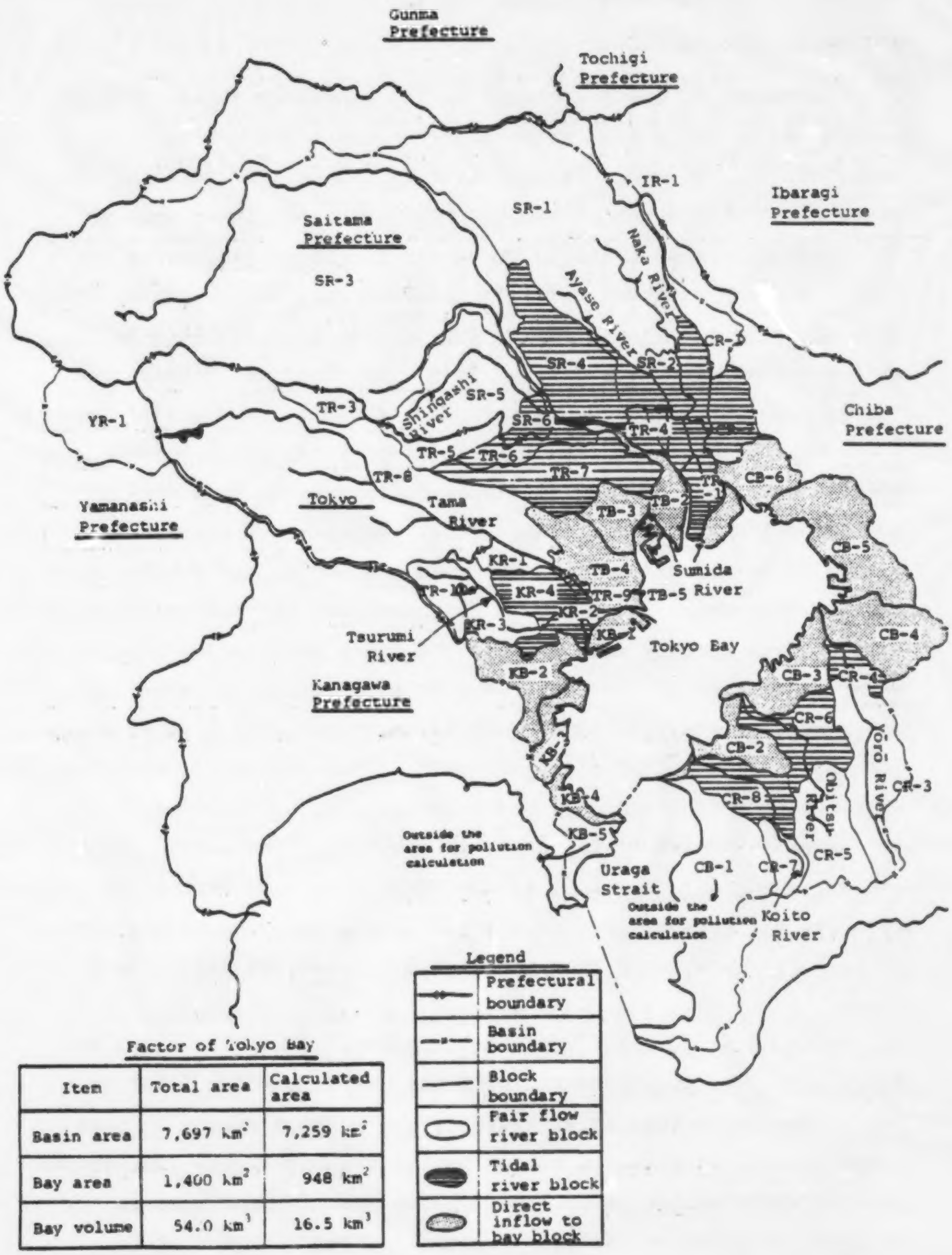
1.1 Natural Conditions

Tokyo Bay is located nearly in the middle of Japan. It is surrounded by the Miura Peninsula to the west and the Boso Peninsula to the east. It is linked to the Pacific Ocean by the Uraga Strait. The term Tokyo Bay designates the inlet area to the north of the Kannon and Futtsu capes. It extends about 80km from the north to south end. Its maximum east-west width is 30km, with an average of about 16 km. The surface area is 1,400 km^2, with a volume of about 54 km^3. To the north of the Honmoku and Futtsu capes, the sea bed is composed of estuarive deposits, with a shallow water depth of less than 40 m and a relatively monotonous bottom configuration. A 1∿2 m shallow intertidal zone extends for about 1∿2 km from the shore line. Water depth increases gradually toward the offing. Between the Kannon and Futtsu capes, the bay narrows to about 6 km. At this line, the bottom configuration differs greatly from that in the inner part of the bay. Water depth also increases to over 50 m, producing a large submarine valley starting at a water depth of 100 m at a point about 2 km off Kurihama in the Uraga Strait. This valley curves point at the bottom of Sagami Bay with a water depth of 1,000 m.

The basin area of Tokyo Bay is about 7,700 km^2. The basin has a rectangular form long in the NW-SE direction across the bay, but the greater part of the basin area strectches to the NW (Fig. 1.1). The basin area is about 5.5 times the bay surface area.

Nine major rivers flow into Tokyo Bay. These are cited in Table 1-1. The precipitation over the basin is about 1,500 mm a year. The Basin area is situated in the monsoon region so that there is much rainfall in summer but little in winter. The annual peak rainfall occurs before the typhoon season (September to October), followed by the rainy season. River flow varies in

Fig. 1.1 Tokyo Bay Basin Map


Gunma Prefecture
Tochigi Prefecture
Ibaragi Prefecture
Saitama Prefecture
Chiba Prefecture
Yamanashi Prefecture
Tokyo
Kanagawa Prefecture
IR-1
SR-1
SR-2
SR-3
SR-4
SR-5
SR-6
Naka River
Ayase River
Shingashi River
CR-1
CR-2
CR-3
CR-4
CR-5
CR-6
CR-7
CR-8
YR-1
TR-3
TR-4
TR-5
TR-6
TR-7
TR-8
TR-9
TR-10
TR-1
TB-2
TB-3
TB-4
TB-5
CB-1
CB-2
CB-3
CB-4
CB-5
CB-6
Tama River
Sumida River
KR-1
KR-2
KR-3
KR-4
KB-1
KB-2
KB-3
KB-4
KB-5
Tsurumi River
Tokyo Bay
Oro River
Obitsu River
Koito River
Uraga Strait
Outside the area for pollution calculation
Outside the area for pollution calculation
Legend
Prefectural boundary
Basin boundary
Block boundary
Fair flow river block
Tidal river block
Direct inflow to bay block

Factor of Tokyo Bay

Item	Total area	Calculated area
Basin area	7,697 km^2	7,259 km^2
Bay area	1,400 km^2	948 km^2
Bay volume	54.0 km^3	16.5 km^3

accordance with the rainfall pattern. Large rivers flowing into Tokyo Bay are heavily utilized for urban water systems. Their water is controlled in a complicated manner, which includes dam operations, introduction of water from other water systems to the north and west and water conveyance between rivers. Thus, because of such artificial controls, river flow into the bay considerably differs from the resulting from natural conditions.

Table 1-1 Major Rivers Flow into Tokyo Bay

	Basin area (km^2)	Inflow into the Bay, 1975 (10^6 m^3/year)	Water quality (BOD_5)	
			1977 mean	Environmental standard
Edo River	183	3,087	3.3	B(3)
Naka River	915	1,145	6.4	C(5)
Ara River	2,267	821	2.9	B(3)
Sumida River	593	2,042	5.2	D(8)
Tama River	1,184	568	7.3	C(5)
Tsurumi River	238	686	4.4	D(8)
Koito River	146	137	4.0	C(5)
Obitsu River	273	252	3.7	B(3)
Yōrō River	247	218	1.7	B(3)

Numerical figures according to Comprehensive Basin-Wide Sewarage System Study in Tokyo Bay Area.

1.2 Social Conditions

Tokyo Bay basin belongs to the South-Kanto district. Administratively, it includes the Tokyo Metropolis, the greater part of Saitama Prefecture and parts of Kanagawa and Chiba Prefectures. These form the Metropolitan area with the capital Tokyo at its center. It is the center in all aspects of the administration, economy and culture of Japan.

To represent the national status of the Tokyo Metropolis, which is a particularly important administrative entity in this district, the statistics can be given:

1% of agriculture;

10% of population, dwelling houses and social capital;

20% of productive, cultural and consumptive activities;

40% of financial activities; and

60% of the highest executives, accumulation of large enterprises and concentration of intellectuals.

The population and industrial shipment of those parts of the said district which are included in the Tokyo Bay basin in terms of their proportion to those of the whole country in 1975 are given as shown in Table 1-2-1. It will be seen that within the basin, which is only 2% of the whole national land area 20% of the population and 22% of the industrial production of the whole country are conentrated. In Japan, which has the highest density of population and where productive activities are carried out at the highest density in the world, the Tokyo Bay basin is an area which has them accumulated to the highest extent.

Table 1-2-1 Population and Industrial Shipments of the One Metropolis and Three Prefectures of the Tokyo Bay Basin (1975)

Item / Prefecture	Area (km^2)		Population (1,000 persons)		Industrial Shipments (¥100,000,000)	
	Adminis-trative section	Within basin	Adminis-trative section	Within basin	Adminis-trative section	Within basin
Tokyo	2,145	1,724	11,669	11,554	114,866	114,688
Kanagawa	2,391	508	6,398	3,435	120,329	72,487
Saitama	3,799	3,369	4,821	4,551	52,006	47,037
Chiba	5,115	1,910	4,149	2,548	55,840	46,129
Total	(3.6%) 13,450	(2.0%) 7,511	(24.2%) 27,037	(19.7%) 22,088	(26.9%) 343,041	(22.0%) 280,341
Whole Country	377,535		111,934		1,274,329	

Population of administrative sections by National Census; Industrial shipments by Industrial Statistics; and figures within basin by Comprehensive Basin-Wide Sewerage System Study in Tokyo Bay Area.

Under the Third Comprehensive National Development Plan, plans have been made to distribute the population and industrial activities to the local districts so that the weight of the Tokyo Bay basin area to the whole country will relatively decrease in the future. Yet, the population and industrial activities in the Tokyo Bay basin will continue to remain at a high level (Table 1-2-2).

Table 1-2-2 Changes in the Population in the One Metropolis and Three Prefectures

Prefecture	1960		1965		1970		1975	
	Total	Urban area	Total	Urban area	Total	Urban area	Total	Urban area
Saitama	2,431	896	3,015	1,320	3,867	2,126	4,821	3,113
Chiba	2,306	663	2,702	1,053	3,366	1,706	4,149	2,394
Tokyo	9,684	8,908	10,869	10,099	11,408	10,876	11,674	11,279
Kanagawa	3,443	2,411	4,431	3,175	5,472	4,290	6,398	5,401
Total	17,864	12,878	21,017	15,647	24,113	18,998	27,042	22,187

Within the Tokyo Bay basin there are a total of 171 municipalites: 23 special wards, 25 cities, 5 towns and 1 village in Tokyo; 4 cities including 2 specially designated cities in Kanagawa; 39 cities, 35 towns and 16 villages in Saitama; and 17 cities, 10 towns and 1 village in Chiba.

Looking over the locations of these cities, many of the cities located along the bay have an industrial character. Large scale plants such as steel and chemical industry are located along the bay. Further, the bay is surrounded by leading trade ports such as Yokohama Port (annual cargo handling of 116,990,000 tons), Tokyo Port (58,540,000 tons) and Chiba Port (124,820,000 tons). The cargo handling of these three ports accounted for 11% of the whole country. The inland cities are characterized in cocentric circles about the urban center of Tokyo. The urban center of Tokyo is a central area of business, and the resident population is lower than that of its periphery. Some sub-centers are located in the periphery. These center and subcenters are closely connected with one another by means of road and railroad networks. Further out from the sub-centers are located the cities of the so-called "bedroom towns", surrounding the central area. These cities include some new residential towns with populations at the level of about 300,000. The outer periphery of these residential cities extends as far as 50 km from the urban center along railroads running out from the center. The area within the radius of 50 km is rapidly becoming urbanized, and the urabn area is expanding, linking the cities. Beyond the 50 km zone, independent cities are located here and there, but the weight of the population, etc. is very low, if taken against the basin as a whole.

Among the cities within the Tokyo Bay area those performing sewage treatment as of 1978 are 40 in total (with the 23 wards of Tokyo taken as one city), and the number of sewage treatment plants is 58. The population covered by these treatment plants is 9,000,000 or about 40% of the population in the basin area. The total quantity of sewage treated is 7,136,000 m^3/day. The treatment facilities now operating are largely of a combined type, but those to be constructed hereafter are, for the greater part, of separate systems. The urban area is spreading to connect the cities with one another, and this trend will continue or intensify. Thus, for sewage disposal to meet the requirements of such a situation, regional sewage systems are being constructed or planned. In some of the cities in the basin where no sewage treatment is provided, construction of sewage systems is in progress. In almost all of the cities, construction plans are being formulated.

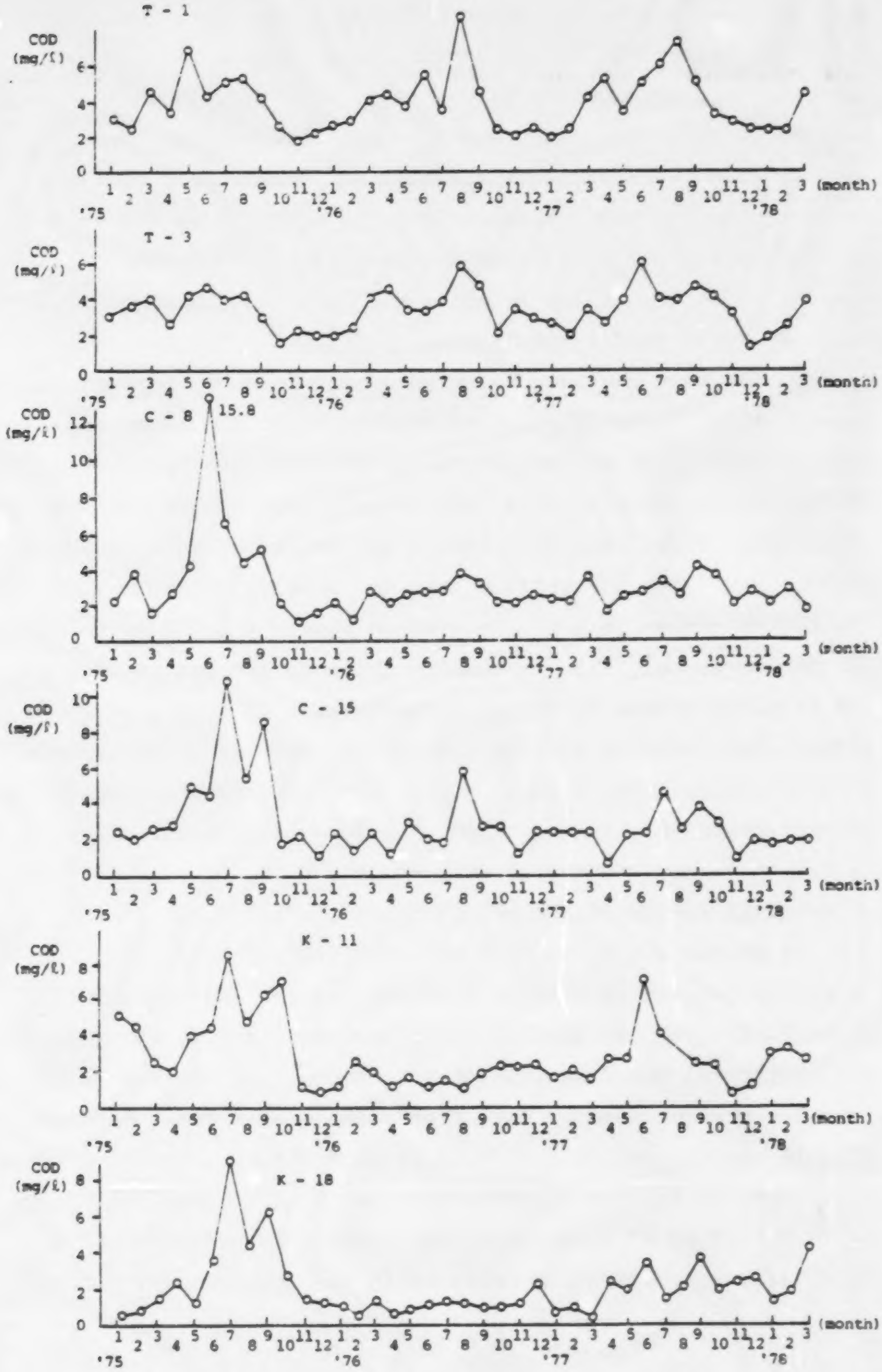
2. WATER QUALITY

2.1 Present Condition of Water Quality

In the Tokyo Bay area artificial pollution is produced in large quantities because of the accumulation of a high density of population and productive activities in the basin on one hand, and from physical factors such as the poor exchange of sea water with the open sea on the other. Thanks to measures taken from the first half of 1970's to control water pollution, the pollution has been reduced in these years. Yet, Tokyo Bay is still highly polluted. From 1971 water analysis has been made periodically in Tokyo Bay. Measurements are made at least once a month at 48 points in the bay. Measured COD mn, which shows the extent of organic pollution, is changing remarkably from season to season. That is, at any point in the bay, the COD mn concentration is low in winter and high in summer. At some locations, it is not rare that the peak value in summer is several times the normal value. This is produced because Tokyo Bay is a water area of considerable eutrophication, so that algae production of phytoplankton constitutes a considerable part of the organic pollution. Thus, seasonal changes are caused by such internal production. This is verified from the vertical distribution of the COD mn concentration. In surface water, the COD mn concentration is very high in summer, while in the bottom layers, the seasonal change in COD mn concentration is negligible. In summer, sea water in the bay stratifies, and the water temperature in the surface layer becomes high. This induces the propagation of phytoplankton. This is also the cause of seasonal changes in COD mn values. Seasonal changes in the COD mn concentration at selected points in the bay is shown in Fig. 2.1.

As stated above, the COD mn concentration in Tokyo Bay has a certain pattern related to seasonal change. In horizontal distribution, it has some of these characteristics. For example, the horizontal COD distribution is not constant throughout the year but changes more or less from season to season. Seasonal wind in the bay may be the cause of such change. In the horizontal distribution, the COD mn concentration is, of course, very high near the influx of large pollution loads. In the case of Tokyo Bay, large rivers are concentrated in the northwestern part of

Fig. 2.1 Seasonal Change of COD mn Concentration
(Mean Value of Surface and Bottom Layers)



the bay. The pollution load is concentrated these. Thus, as a whole, a high concentration area of COD mn appears in the northwestern part of the bay, and the concentration tends to decrease toward the east coast and bay entrance to the south. The influence of seasonal winds appears to be strong, particularly in the surface layer. The southerly wind in summer acts to prevent the high concentration area in the north from spreading into the bay, while the northerly wind in winter acts conversely to accelerate the diffusion. Distributions of COD mn concentration in February and May 1975 are illustrated in Figs. 2.2 and 2.3, respectively.

2.2 Influent Pollution Load

For the simulation of water pollution in Tokyo Bay and the examination of policy measures for reducing the load and meeting environmental standards, a forecast of the pollution load occurring in the basin and that flowing into Tokyo Bay was made.

Pollution loads from artificial sources, such as domestic sewage, industrial wastewater and livestock wastewater, were calculated by multiplying the population, industrial shipment and quantity of poultry by the respective unit values. The generated load is treated at sewage treatment plants, night-soil disposal plants or treatment facilities provided by factories to meet the effluent standards before it is discharged into a river, etc. The discharged COD mn load into the basin is about 700 tons/day or 35% of the total generated (about 2,000 tons/day).

The loads discharged into the basin flow into Tokyo Bay via rivers or other water channels. The routes differ. In the case of Chichibu City, located at the western part of the basin, the load flows a distance of nearly 150 km. On the other hand, sewage treatment plants located along the bay coast discharge the load directly into Tokyo Bay. Such routes of flow into the bay were charted properly, and the influx to Tokyo Bay was calculated according to the method described below. For the load flowing

Fig. 2.2 Distribution of COD mn Concentration in February 1975
(Mean Value of Surface and Bottom Layers)

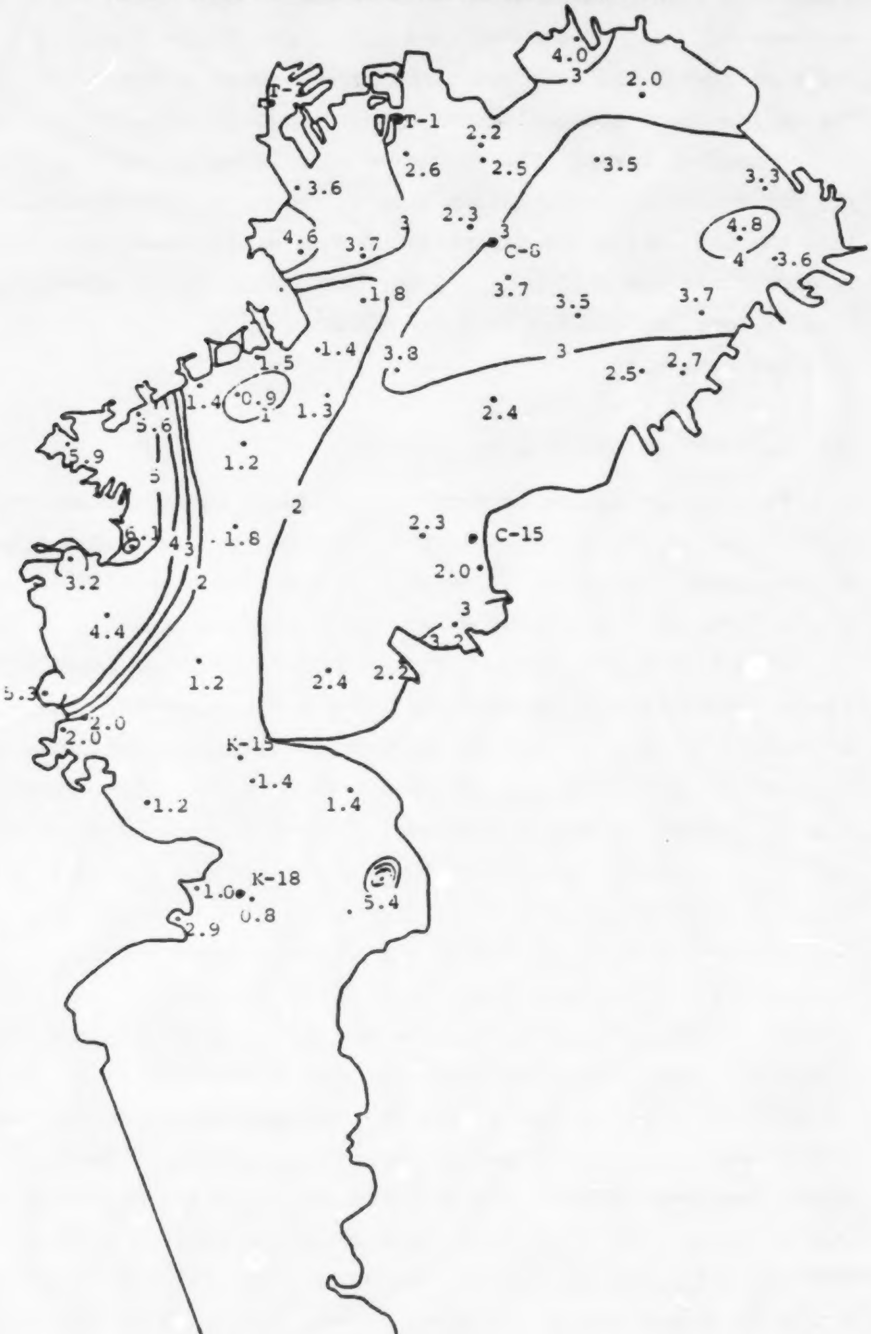
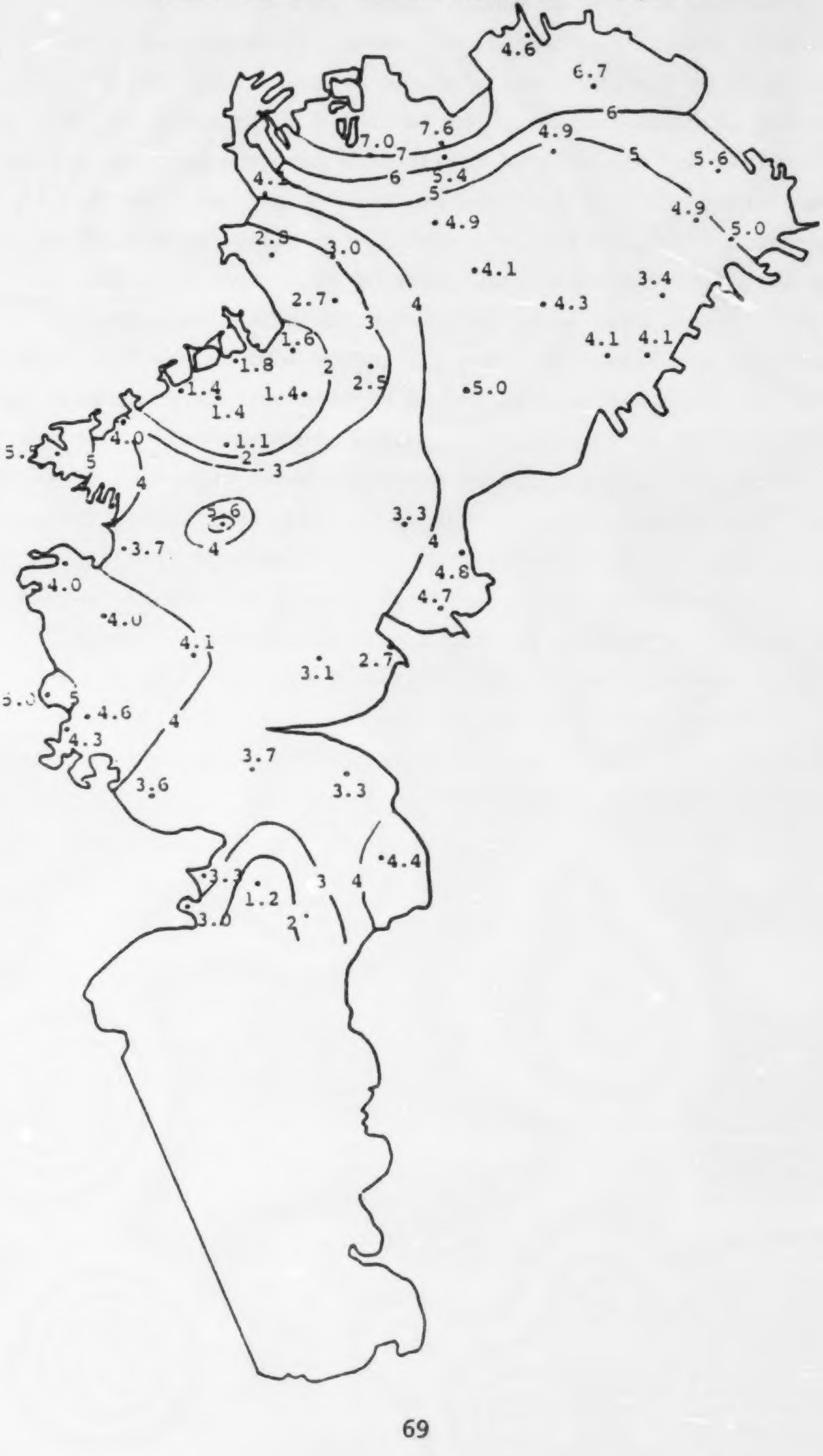
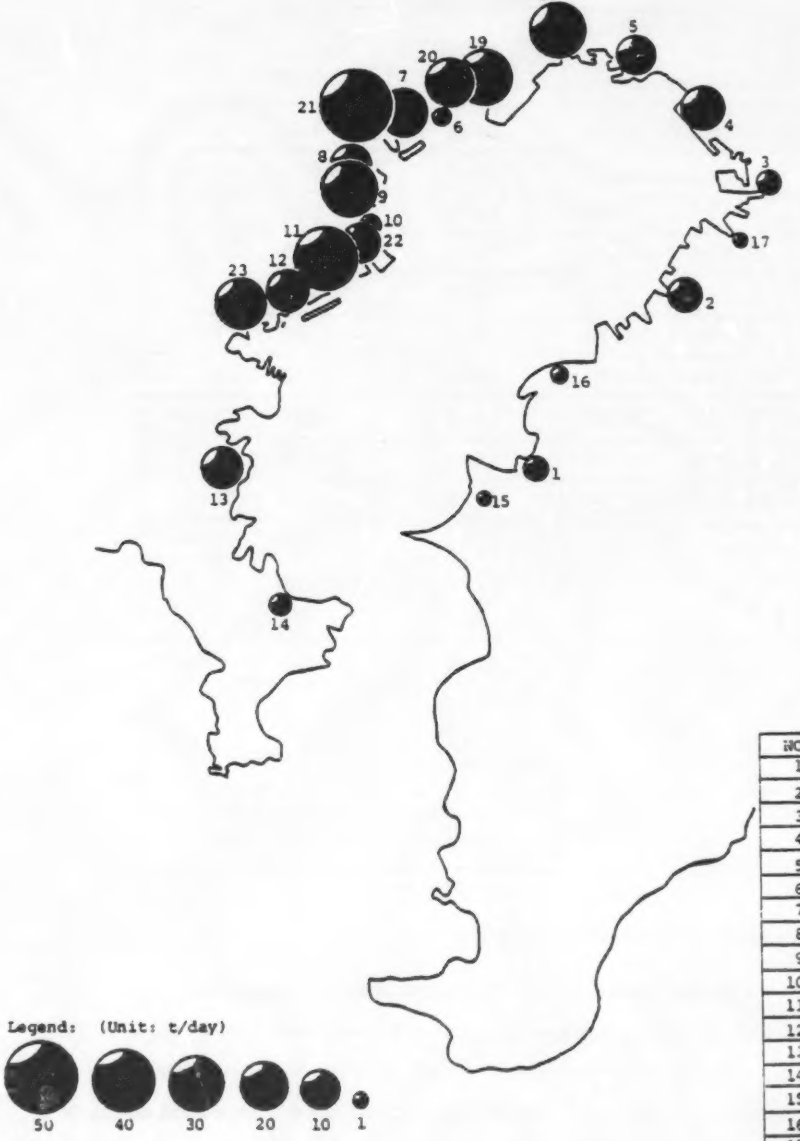

4.0
3
2.0
T-2
T-1
2.2
2.6
2.5
3.5
3.3
3.6
2.3
3
3
4.8
4.6
3.7
C-6
4
3.6
1.8
3.7
3.5
3.7
1.4
3.8
3
2.5
2.7
1.5
1.4
0.9
1
1.3
2.4
5.6
5.9
1.2
5
2
2.3
C-15
6.1
4
3
1.8
2.0
3.2
2
3
4.4
3.2
1.2
2.4
2.2
5.3
2.0
2.0
K-15
1.4
1.4
1.2
K-18
1.0
0.8
5.4
2.9

Fig. 2.3 Distribution of COD mn Concentration in May 1975
(Mean Value of Surface and Bottom Layers)


4.6
6.7
6
7.0
7.6
7
4.9
5
5.6
6
5.4
5
4.1
4.9
4.9
5.0
2.9
3.0
4.1
3.4
2.7
4
4.3
3
1.6
4.1
4.1
1.8
2
2.5
5.0
1.4
1.4
1.4
4.0
1.1
5.5
5
2
3
4
3.3
5.6
4
3.7
4
4.8
4.0
4.7
4.0
4.1
2.7
3.1
5.0
5
4.6
4
4.3
3.7
3.6
3.3
4.4
3.3
3
4
1.2
3.0
2

into Tokyo Bay via a rivers, the lowermost point of the river at which both flux and COD mn concentration were obvious (such points were termed 'the fair flow terminals' was plotted. The transit load at the point was obtained by multiplying the flux by the COD mn concentration. For the river basin above the fair flow terminal and the area of direct flow into the bay, the influx into the river and that into the bay were calculated from the discharged loads according to a specific formula. This formula was derived from the measurement data of six areas, with the population density and water volume density of the respective basins taken as parameters. For the inflow load to the fair flow terminal of the river, influx to the bay was calculated by measurements of the COD mn concentration in the river channel. When sea water was mixed, dilution by sea water was taken into consideration in the value of measurement of the $C\ell^-$ ion concentration. Further, in the rivers, the COD mn concentration was measured at various points at a frequency of at least once a month so that the influx to the bay was calculated by month. The annual mean values of the influx load and influx water volume (fresh water) to Tokyo Bay in 1975 are given in Table 2-2 and Fig. 2.4. The influx load to the bay is about 440 tons/day or nearly 63% of the approximately 700 tons/day total load discharged in the basin.

Fig. 2.4 Distribution Chart of COD Load Flowing into the Bay


18
5
19
20
7
21
6
4
8
3
9
10
11
22
17
12
23
2
16
1
13
15
14
Legend: (Unit: t/day)
50
40
30
20
10
1

NO.	Nomenclature
1	CB-2
2	CB-3
3	CB-4
4	CB-5
5	CB-6
6	TB-1
7	TB-2
8	TB-3
9	TB-4
10	TB-5
11	KB-1
12	KB-2
13	KB-3
14	KB-4
15	Koito River
16	Obitsu River
17	Yōrō River
18	Edo River
19	Naka River
20	Ara River
21	Sumida River
22	Tama River
23	Tsurumi River

Table-2-2 Influx Water Volume and Load into Tokyo Bay

Units: Water Volume, 1,000 m^3/day; and
Load, COD mn t/day

Block	1975 Mean		Block	1975 Mean	
	Water Volume	Load		Water Volume	Load
Koito River	374	0.66	CB-2	468	4.13
Obitsu Rvier	691	1.42	CB-3	533	6.14
Yōrō River	597	0.70	CB-4	514	4.20
Edo River	8,458	46.00	CB-5	983	12.64
Naka River	3,138	41.72	CB-6	628	11.61
Ara River	2,248	29.92	TB-1	26	1.44
Sumida River	5,595	82.90	TB-2	641	22.47
Tama River	1,557	10.68	TB-3	983	22.18
Tsurumi River	1,879	19.86	TB-4	1,575	40.29
River Total	24,537	233.86	TB-5	54	2.74
			KB-1	1,768	41.44
			KB-2	674	19.91
			KB-3	778	14.38
			KB-4	204	3.29
			Direct discharge to the Bay Total	9,829	206.86
			Total	34,366	440.72

2.3 Water Quality Simulation Models

Analysis of water pollution in Tokyo Bay by computer was started in or about 1970. Some models are now available in the respective fields. There are two water quality simulation models in use. One is a tidal current simulation model for the reproduction of tidal phenomena in the bay with the water level variation at the bay entrance, inflow to the bay, etc. given. The other is a water quality simulation model reproducing the distribution of COD mn concentrations in the bay from the inflow, advection, diffusion decrease, etc. of the load in use with the results of tidal calculations. The models used this time are outlined below.

Tidal Current Simulation Model

- Two dimensional, one layer; Tokyo Bay sectioned in 2 km-meshes.
- Partial differential equations of the equations of continuity and motion are substituted by differential equations, and numerical solutions are obtained under given initial and boundary conditions.
- The result is expressed as a value of integration of the tidal current in one day, or the so-called residual flow.

Water Quality Simulation Model

- Two dimensional, one layer; The regions of calculation and mesh are the same as those of the tidal current calculation model. In the vertical direction, water quality is assumed to be uniform.
- Change in water pollution quantity in the respective meshes is resolved by numerical analysis according to the so-called equation of diffusion, under which the change is due to advection, diffusion, inflow of the load and decreases owing to self-purification.
- For tidal currents, the results of the calculations from the tidal current calculation model, or the residual flow, is used. Accordingly, the result is given as daily mean water quality.

The results of reproduction of the average residual flow in the bay in May according to the tidal current calculation model and of the COD mn concentration (except COD mn by algae production) in the bay, according to the water quality calculation model, are shown in Figs. 2.5 and 2.6, respectively. The results of reproduction of both tidal current and COD mn concentration are considered to be good, upon reference to the results of observation so far obtained.

Fig. 2.5 Tidal Current Calculation Result,
Residual Flow, May 1975

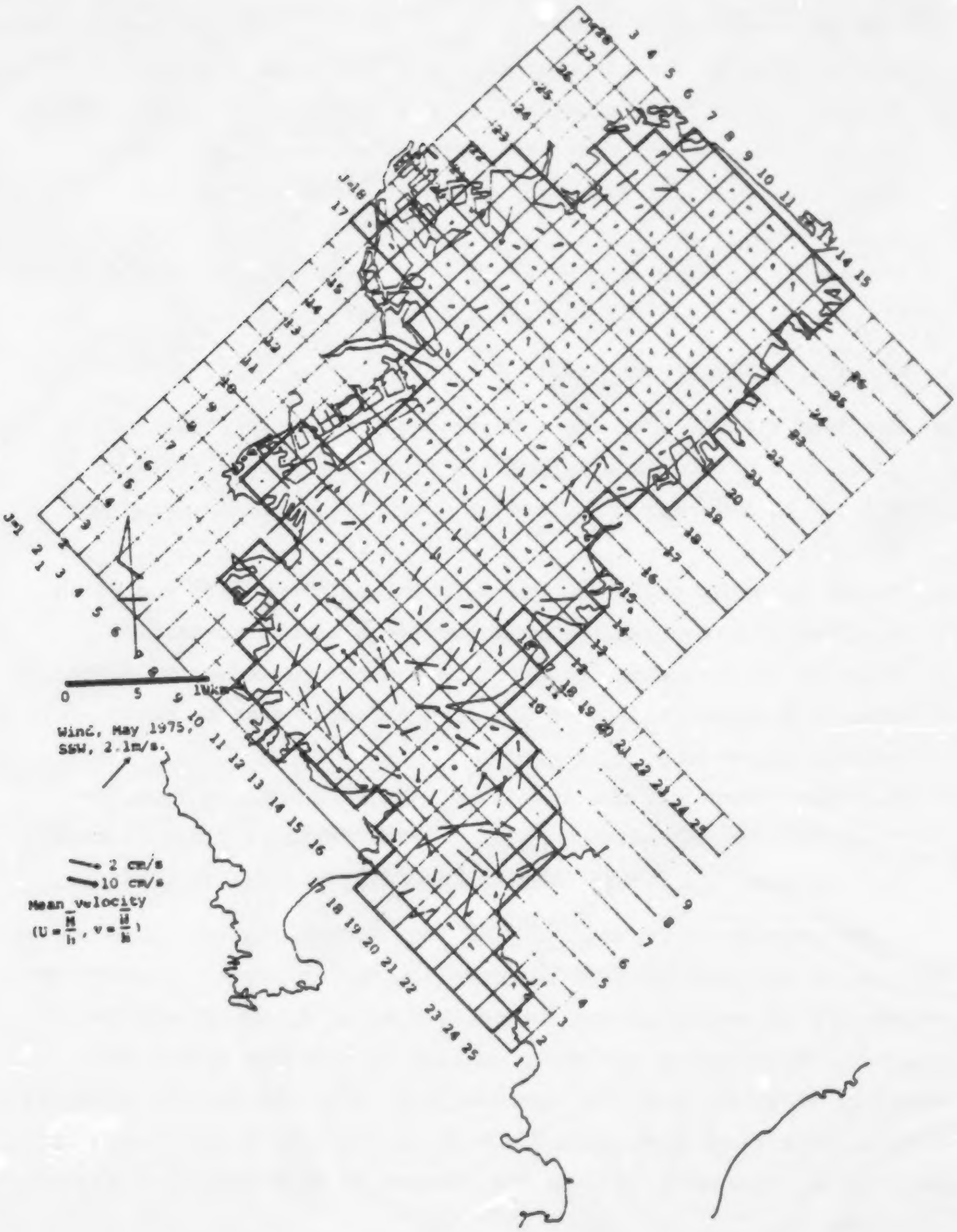
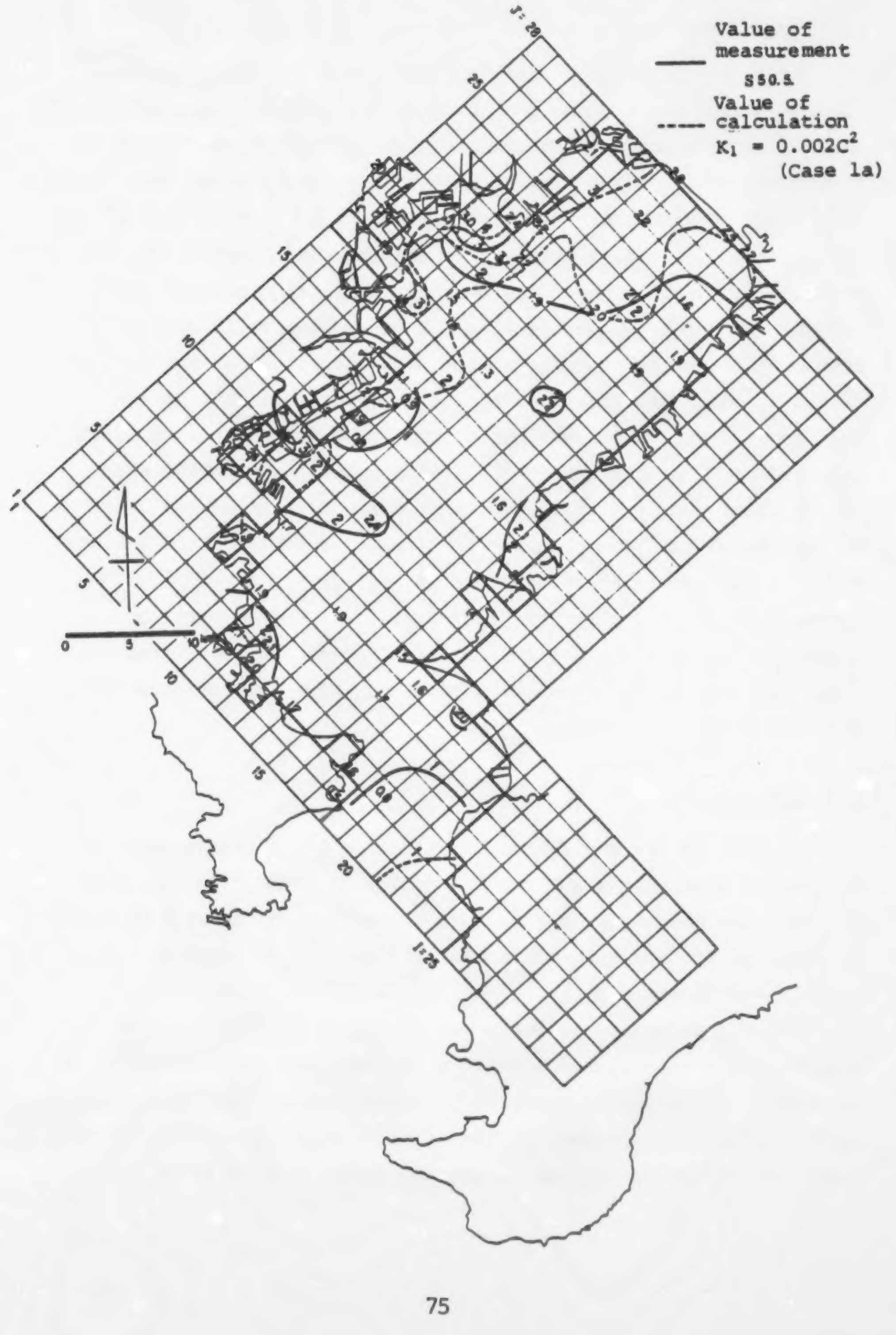

J=1
J=16
J=26
0
5
10km
Wind, May 1975,
SSW, 2.1m/s.
2 cm/s
10 cm/s
Mean velocity
(U = M/h, V = N/h)

Fig. 2.6 Water Quality (COD) Calculation Result, May 1975


Value of measurement
S50.5
Value of calculation
$K_1 = 0.002C^2$
(Case 1a)
J=20
25
15
10
5
1
1
5
10
15
20
I=25
0
5
10

3. ENVIRONMENTAL STANDARDS AND MEASURES FOR THEIR ACHIEVEMENT

3.1 Environmental Standards and Current Water Quality

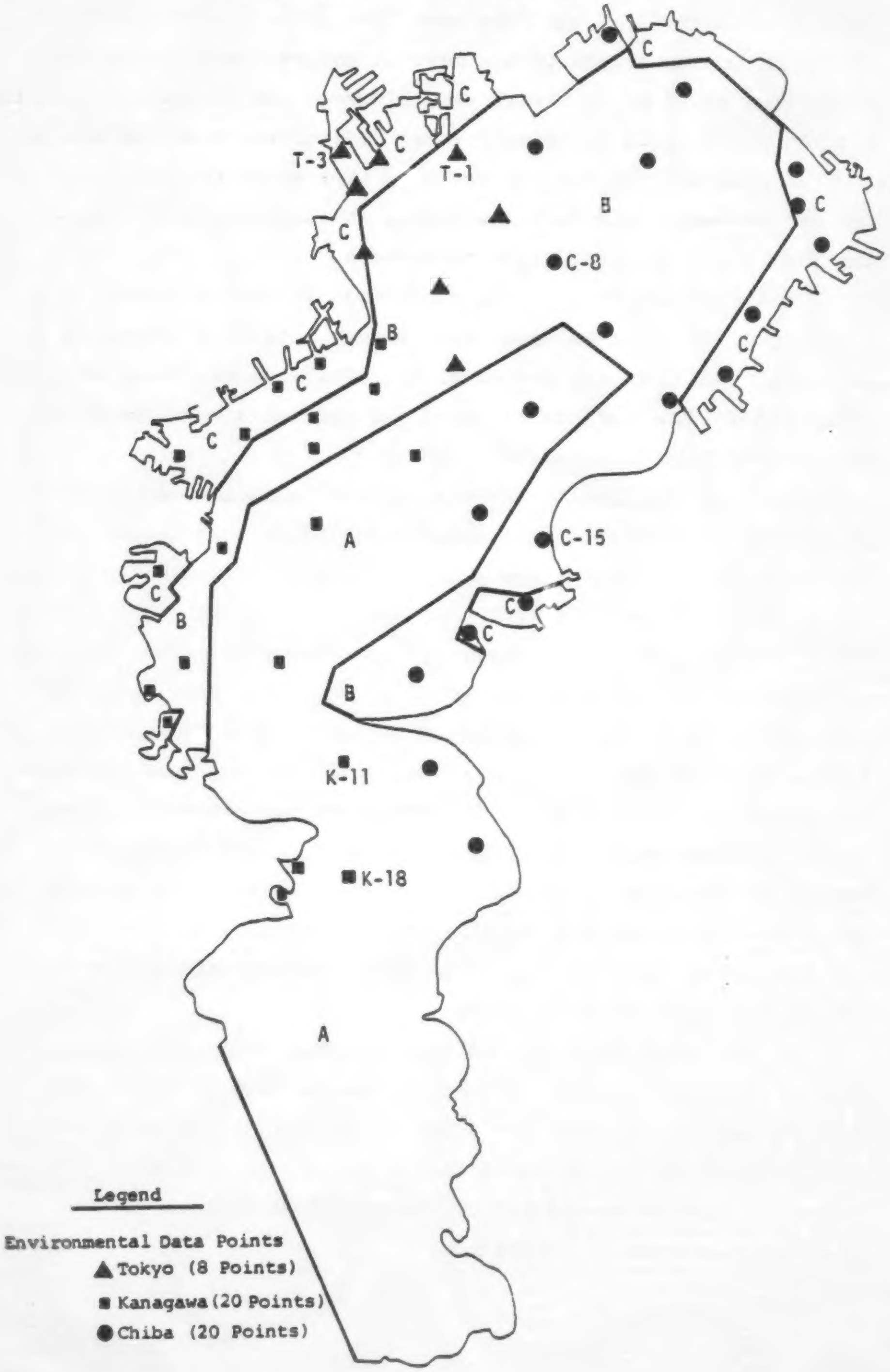
Tokyo Bay is divided into 18 water areas, and for each water area, an environmental water quality standard is stipulated. Taking the environmental standards in terms of COD mn concentration, the bay is classified into 3 categories of A(less than 2mg/ℓ), B(less than 3 mg/ℓ), and C(less than 8 mg/ℓ). To check whether or not the environmental standard is observed, a total of 48 environmental data points are provided in the bay, according to the water area classification, and at each point, measurement of the water quality is made at a frequency of at least once a month. Specifications of the environmental standard and locations of the environmental data points are shwon in Fig. 3.1. Whether or not the environmental standard is achieved is determined by the 75% value of annual measurement of the water quality (or the value corresponding to 1/4 of the whole counted from higher concentration). As of 1975, the environmental standard was achieved at 21 points, or 44% of all environmental data points. However, they are, for the most part, points belonging to Class C along the coast. In Class A and B water areas constituting the greater part of the bay, the environmental standard is rarely achieved.

3.2 Allowable Load

In view of the environmental standards being rarely achieved in Classes A and B against the COD mn influx load of 440 tons/day of 1975, the extent the influx load to the bay should be suppressed in order to achieve the environmental standards of Tokyo Bay as a whole was determined.

Organic pollution of Tokyo Bay is caused by two factors: pollution due to influent organic pollutants and that due to algae production by phytoplankton. In principle, these two are considered to be independent phenomena. The highest frequency of the COD mn concentration corresponding to the 75% value as a whole of the bay

Fig. 3.1 Water Quality Environmental Data Points and Class Specifications in Tokyo Bay


C
C
T-3
C
T-1
B
C
C
C-8
B
C
C
C
C
A
C-15
C
C
B
C
B
K-11
K-18
A
Legend
Environmental Data Points
▲ Tokyo (8 Points)
■ Kanagawa (20 Points)
● Chiba (20 Points)

is May. When this period is taken, the influent load accounted for 47% of the COD mn of the bay in May 1975 and internal production accounted for the remaining 53%. From the distribution of COD mn concentration in the bay, it was presumed that algae production would be of higher weight toward the central part of the bay which had a relatively lower concentration of COD mn.

In such a situation, it would be imperative in achieving the environmental standard, to reduce the influent load and, at the same time, suppress algae production.

The mechanism of internal production is not yet known, although efforts are being made to clarify it. The numerical simulation model is not yet complete. Thus, in obtaining the allowable influx load, calculation was made with some stages set for the level of internal production.

From the calculation results, it was found that in order to achieve environmental standards, the influx load had to be less than about 220 tons/day and that algae production had to be suppressed to a level of less than about 1/4 of that in May 1975. The allowable quantity of about 220 tons/day corresponds to about one half of the inflow of 440 tons/day in 1975. When the storm water overflow from the existing combined sewer systems in the basin, to which control measures are difficult to apply, or natural pollution due to storm water is considered, considerable efforts would be required for reducing the current inflow by more than one half. When the increasing quantity of water to be treated in the future is taken into consideration, treatment facilities in the basin must have the discharge water quality improved to about COD mn 5-10 mg/ℓ after 20 years.

On the other hand, in order to suppress algae production to that of about 1/4 of the present, if the factors governing the rate of algae production are taken to be nutrient salts of nitrogen and phosphor, it is estimated that a greater part of this generation will have to be reduced by the discharge control at the artificial sources of generation.

4. PRESENT STATUS AND PROBLEMS OF SEWERAGE DEVELOPMENT AND SEWAGE SLUDGE TREATMENT AND DISPOSAL IN TOKYO BAY AREA

Development of sewerage systems in Tokyo Bay area is in progress mainly around the large cities, and in the 23 wards. The development is complete for 70% of the whole area. In the peripheral areas, 2 or 3 cities have almost completed the development. However, in Chiba and Saitama, where population is increasing rapidly, development is slow. Thus, total coverage by sewerage systems in the area as a whole is about 40 percent.

Construction of sewerage systems is being urged for various sites in the area, and in Tokyo, Yokohama, Kawasaki and Chiba cities and Tokyo, Saitama and Chiba Prefectures, construction of regional sewerage systems designed for smaller cities in the area is in progress.

The cost of construction of the sewerage systems in the area around Tokyo Bay is about ¥480,000,000,000 (or about $2,000,000,000) in fiscal 1979.

In regard to drainage systems, in those areas of Tokyo, Kawasaki and Yokohama where sewerage was planned and construction was executed from an early time because of the importance of the area for the respective cities or as a measure for preventing storm water flooding, the combined system is extensively employed. But, in the areas where construction has started recently, separate systems are generally employed.

Table 4-1 shows the major sewage treatment plants now in operation or under construction.

At present, the largest plant is the Morigasaki Wastewater Treatment Plant, Tokyo, located near the former Tokyo International Airport.

Treatment is conducted largely under the activated sludge process, and the sludge is subjected to dewatering directly or after anaerobic digestion. It is then disposed of, partly after incineration or by land reclamation.

Table 4-1 Major Sewage Plants in Operation or under Construction (Tokyo Bay)

Planned treatment capacity, 200,000 m^3/day or more.

Name of Plant	Type and Project Entity		Starting Date of Operations	Current Capacity	Planned Capacity	Remarks
[Saitama Prefecture]				1,000 m^3/day	1,000 m^3/day	
Arakawa Left Side Basin, South	Regional Sewerage	Saitama Prefecture	1972	210	1,383	8 Cities.
Arakawa Left Side Basin, North	"	"	-	-	602	5 Cities, 1 town.
Arakawa Right Side Basin	"	"	-	-	1,280	10 Cities, 3 towns.
Nakagawa	"	"	-	-	2,040	9 Cities, 8 towns.
[Chiba Prefecture]						
Inbanuma Basin	Regional Sewerage	Chiba Prefecture	1974	154	1,344	7 Cities, 5 towns, 2 villages.
Edogawa, Left Side Basin	"	"	-	-	1,524	6 Cities, 1 town.
Chiba City, South	Municipal Sewerage	Chiba City	-	-	301	
[Tokyo Metropolis]						
Shibaura	Municipal Sewerage	Tokyo Metropolis	1931	1,130	1,590	
Mikawajima	"	"	1922	700	950	
Kosuge	"	"	1977	150	450	
Sunamachi	"	"	1930	680	1,220	
Odai	"	"	1962	358	420	
Shingashi	"	"	1974	705	1,350	
Ochiai	"	"	1964	450	450	
Morigasaki	"	"	1966	1,280	1,810	
Kitatama No. 1	Regional Sewerage	"	1973	136	450	6 Cities.
Tamagawa Upstream	"	"	1976	75	646	5 Cities, 2 towns.
Minamitama	"	"	1971	53	473	5 Cities.
Arakawa Right Side, Tokyo	"	"	-	-	693	10 Cities.
[Kanagawa Prefecture]						
Kohoku	"	Yokohama City	1972	88	439	
Midori	"	"	1977	74	443	
Kanagawa	"	"	1978	191	517	
Nambu	"	"	1965	234	234	
Kanazawa	"	"	1979	69	325	
Iriezaki	"	Kawasaki City	1961	300	357	
Kase	"	"	1973	122	245	
Todoroki	"	"	-	-	360	

*1 Treatment systems all use the activated sludge process.

In order to achieve the environmental standards in Tokyo Bay, advanced wastewater treatment (AWT) will be required in almost all the treatment plants in the future. However, extension of the sewerage systems is of primary importance, so that construction presently is limited to secondary treatment facilities. But, hereafter, construction of AWT facilities must be expedited.

Most treatment plants can only afford enough land for secondary treatment. Thus, securing enough land for wastewater treatment and constructing facilities including those for AWT are problems to be resolved later.

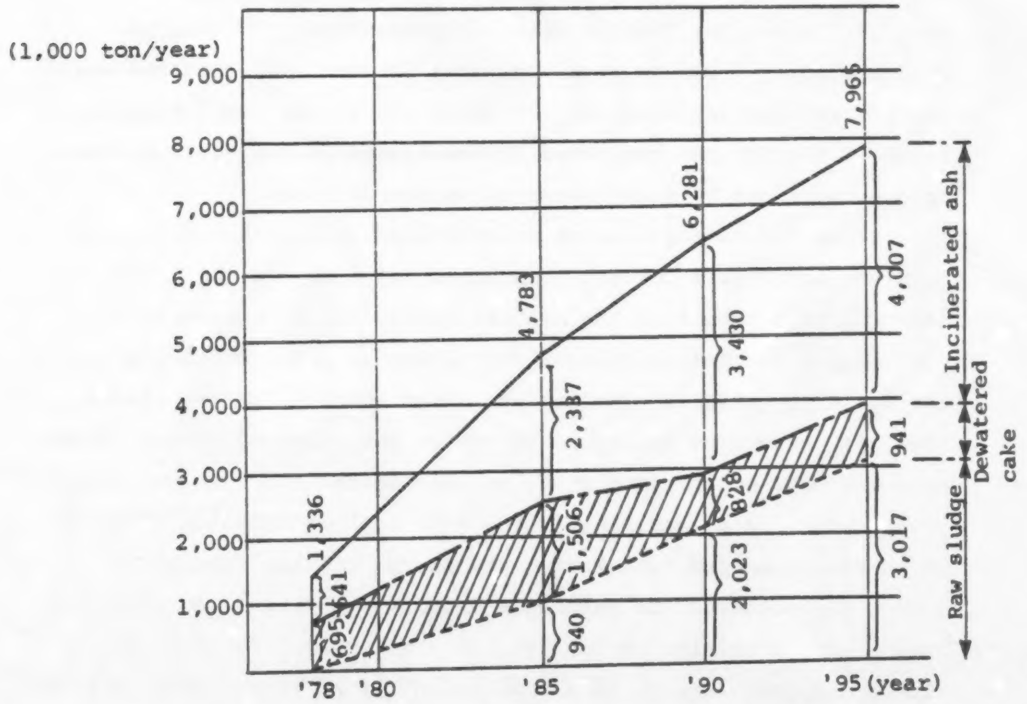
On the other hand, as the sewerage project progresses and treatment becomes intensive, it is believed that there will be increasing quantities of sewage as well as sludge. The result of a survey of the quantities of sewage and sludge at present and in the future is shown in Table 4-2. As seen, in 1995, the population served by sewage treatment will expand about 3.1 times, effluent about 3.8 times and sludge in terms of dewatered cake about 5.9 times the 1978 values. At present, disposition of sewage sludge in the Metropolitan area is such that land and sea surface reclamation accounts for about 99% of the final disposition. Figure 4.1 shows the generated sewage sludge in the Metropolitan area by year and by form (shown as dewatered cake).

If we assume that sludge is generated as a matter of course from the activities for the maintenance of life and the living of man, who is a member of the natural world, it is considered to be a principle for disposition of the sludge to place the sludge in the cycle of material circulation. Particularly, in the inland area, it is thought desirable to review the value of sewage sludge as a resource and replace it in the biological cycle as soon as practicable, in a reasonable form, with soil as a medium to return it to the green and farm land. Studies on the application of sludge for construction materials such as aggregate, reduction to ocean, etc. must also be carried out positively. In this way, sludge disposal should, as a rule, take a form of infinite disposal.

Table 4-2 Sewage Treatment in 1978 and 1985 in the Tokyo Bay Area

Items \ Year	1978	1995	1995/1978
1) Area served (ha)	78,820	299,400	3.8
2) Population served (1,000 persons)	9,003	27,550	3.1
3) Population density (2)/1) 1,000)(persons/ha)	114	92	0.8
4) Quantity of effluent (1,000 m^3/day)	7,136	27,422	3.8
5) Sludge quantity generated (1,000 m^3/day in terms of cake)	1,340	7,970	5.9
6) Number of treatment plants	58	75	1.3
7) Population served per plant (1,000 persons)	155	367	2.4
8) Quantity of effluent per plant (1,000 m^3/day)	123	366	3.0
9) Sludge quantity per plant (1,000 m^3/day)	21	109	5.2

Fig. 4.1 Generation of Sludge by Type and by Year (in Terms of Dewatered Cake)



(Note) Moisture content of dewatered cake = 75%.

But, in the large city regions such as the Metropolitan area where the urban area is expanding continously and the population is overcrowded, for almost all cities it is difficult to secure land for treatment plants to dispose of sludge independently. For the adequate disposition of the growing amounts of sludge generated hereafter, it is very difficult to complete the disposition of the sludge within the administrative area of one local government. It is, therefore, required for local governments concerned to plan an area-wide sewerage project conforming to their individual requirements and thus promote the project.

5. OUTLINE OF REGIONAL SEWERAGE PLAN

5.1 Advanced Wastewater Treatment Plan

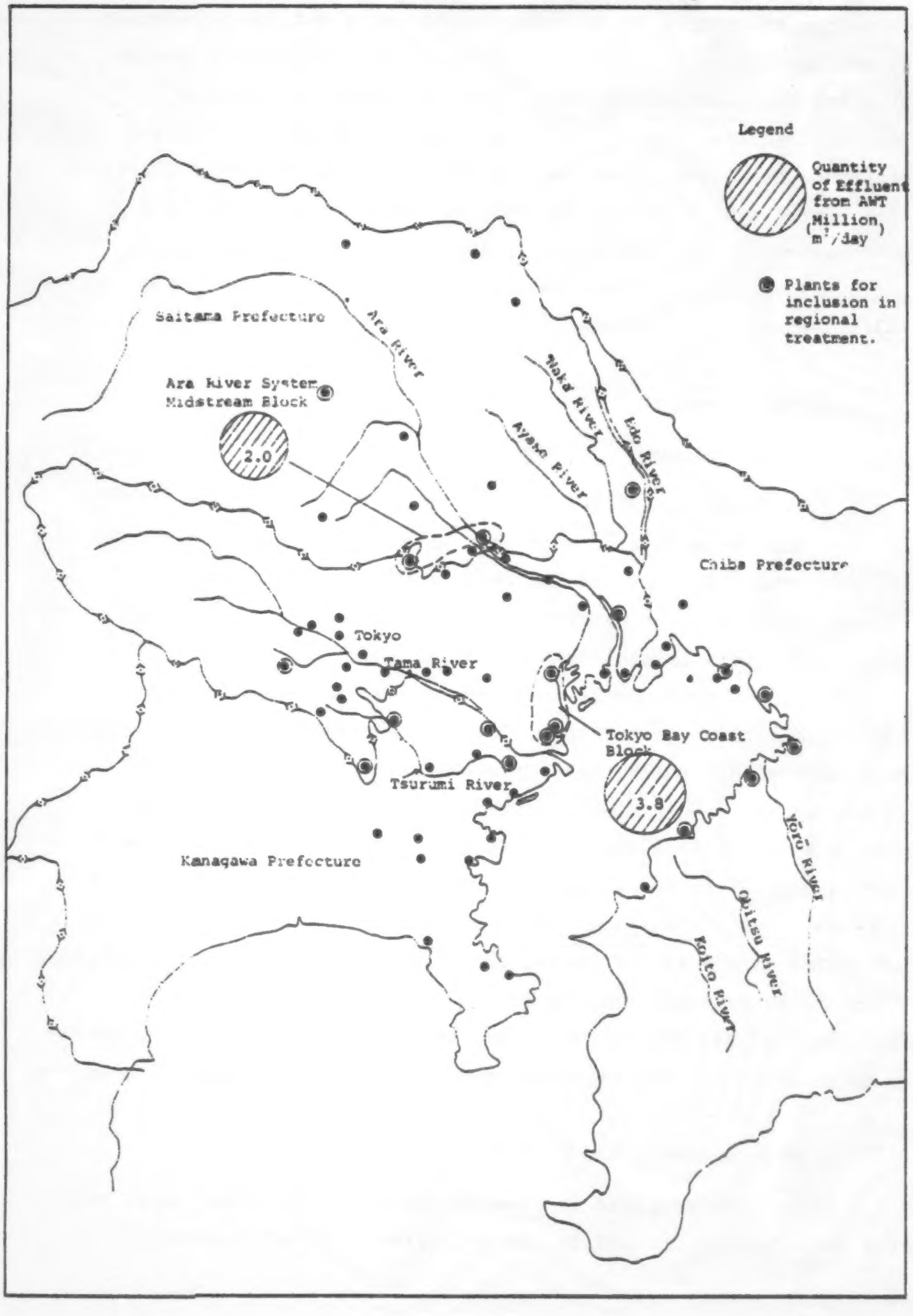
In planning an AWT system for Tokyo Bay, it should be kept in mind that some local governments have their own individual AWT plans. Others do not. In the case that a plan permitting efficient and effective development is developed for the latter, a regional block is conceivable for the AWT.

Thus, it was arranged that the reigonal blocks would be established as shown in Fig. 5.1. It was decided that (1) those treatment plants which could from the economic as well as technical points of view individually perform AWT should be excluded and (2) consideration should be given to recycle the effluent from AWT, and that for the others, individual treatment would be considered. Under this plan, it is estimated that against the flow of effluent from AWT of 27,420,000 m^3/day in 1995, the corresponding quantity of effluent treated by regional AWT plants would be 5,780,000 m^3/day for the Tokyo Bay coast block and Ara River System midstream block. The required land area would be about 130 hectares.

5.2 Sludge Treatment and Disposal Plan

Along with progress in sewerage development on one hand, and the requirements for AWT in the achievement of environmental

Fig. 5-1 Quantities of Effluent from AWT by Block


Legend
Quantity of Effluent from AWT (Million, m³/day)
Plants for inclusion in regional treatment.
Saitama Prefecture
Ara River
Ara River System Midstream Block
2.0
Naka River
Edo River
Ayase River
Chiba Prefecture
Tokyo
Tama River
Tokyo Bay Coast Block
3.8
Tsurumi River
Kanagawa Prefecture
Yoro River
Obitsu River
Koito River

standards on the other, the quantity of sewage sludge will increase in the future. For treatment and disposal of such sludge, it is very difficult to secure land, not only for sewage treatment plants, but also for the disposal of the sludge. The reasons for this are given below.

(1) In the Metropolitan area, land is heavily used, so that it is in fact impossible to secure adequate land.

(2) It is very difficult to obtain the consent and cooperation of inhabitants who live near land designed for disposal.

(3) The disposal land must be sought in the future at more remote sites, and this involves a number of problems such as transportation costs, transportation systems, etc.

For the foregoing reasons, disposal must be directed toward seaside reclamation, etc. However, adequate sites for this are limited. It is, therefore, necessary for local governments to cooperate with one another over a wide area and use the available space for disposition effectively, and thus ensure stable disposal of sludge over a long period in the Metropolitan area as a whole.

5.3 Outline of Regional Sewage Sludge Treatment and Disposal Plan

Tables 5-1 and 5-2 show the scale of the regional age sludge treatment and disposal bases determiend from the future plans of the respective local governments and forecasts of the quantity of sludge generated. These plans include the sludge transportation system, relay stations, sludge treatment facilities (thickener, digestion tank, dewatering equipment and incinerator), landing equipment, return flow treatment equipment, power generation equipment, etc. A schematic diagram of the foregoing is shown in Fig. 5.2.

Table 5-1 Scale of Regional Sludge Treatment Facilities

	Scale of Treatment Facilities m^3/day	Land Required for Facilities ha
Raw sludge	66,540 (DS 998 t/day)	Approx. 50 ha
Dewatered cake	1,910	
Ash	1,370	

Table 5-2 Scale of Regional Sludge Disposal Land

Sea side reclamation	600 ha
Sludge quantity for reclamation	87 mil./m^3

5.4 Effects of Regional Sewage Plan

When sewage treatment/disposal is taken up as a regional project in a large city area, the following advantages may be cited.

(1) An efficient social investment is effected.

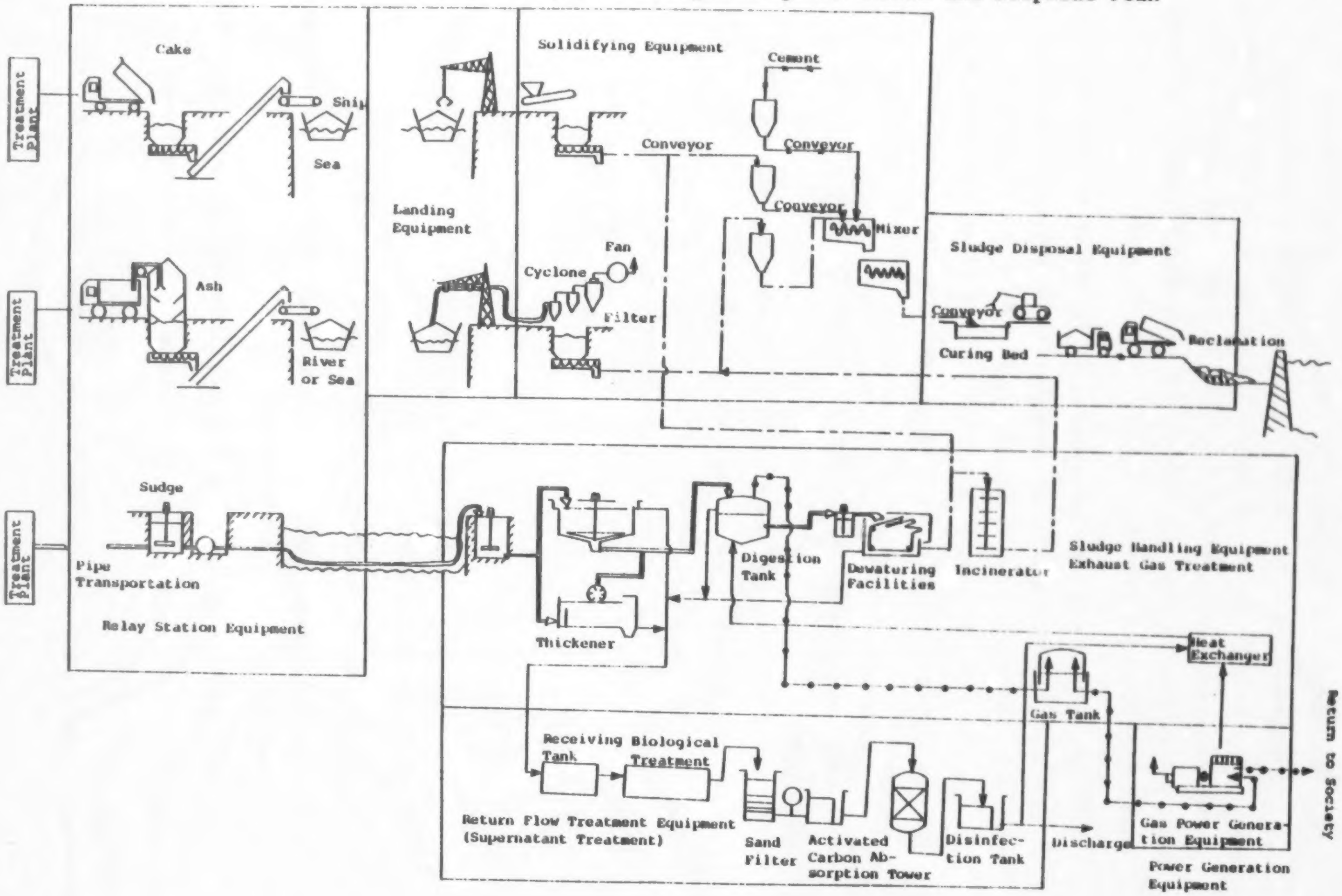
By executing a regional project, social investment becomes more efficient than if the project is executed separately by the local governments concerned. Further, the additional value of creating a new space in a large city area is substantially great.

(2) Adequate maintenance is made.

By extending the project over a wide range, the maintenance cost is reduced. For the maintenance and operation of AWT, excellent engineers can be obtained. This strengthens the maintenance and operation system.

Further, sludge disposition is made systematically under careful management by the regional sludge treatment/disposal projects.

Fig. 5-2 Schematic Diagram of Regional Sewage Sludge Treatment and Disposal Plan


Treatment Plant
Treatment Plant
Treatment Plant
Cake
Ship
Sea
Ash
River or Sea
Sudge
Pipe Transportation
Relay Station Equipment
Landing Equipment
Solidifying Equipment
Cement
Conveyor
Conveyor
Conveyor
Mixer
Fan
Cyclone
Filter
Sludge Disposal Equipment
Conveyor
Curing Bed
Reclamation
Thickener
Digestion Tank
Dewatering Facilities
Incinerator
Sludge Handling Equipment
Exhaust Gas Treatment
Heat Exchanger
Gas Tank
Return to Society
Receiving Tank
Biological Treatment
Return Flow Treatment Equipment
(Supernatant Treatment)
Sand Filter
Activated Carbon Absorption Tower
Disinfection Tank
Discharge
Gas Power Generation Equipment
Power Generation Equipment

(3) Reclaimed land is usable for AWT along the bay coasts.

Considering the very difficult condition for securing of the land for AWT, some treatment plants along the Tokyo Bay coast must use seaside reclamation for land for AWT. The reclaimed land is usable as land for AWT of the secondary treatment effluent gathered from such plants.

(4) The existing sludge treatment land is usable as land for AWT.

Under the very difficult conditions for securing the land for AWT, there are some treatment plants which are capable of coping with AWT if sludge treatment land is converted, with the sewage sludge treatment and disposal resorted to the regional system.

(5) Recovery of the energy from sludge provides an economic return to society.

By recovering the energy from decomposition gas, etc. generated from a large amount of sewage sludge effectively, the energy consumed in the plant can be provided and the excessive energy can be returned to society through power generation, etc. Simultaneously, the quantity of sludge to be disposed of may be reduced.

5.5 Future Problems

Carrying out the plan in the future requires a full examination of the following problems.

(1) Execution of environmental assessment.

(2) Technical development of the processes of energy and resource saving.

(3) Examination of the plan for recycling the effluent from AWT.

(4) Long ranging forecasting of sewage sludge treatment and disposal.

(5) Examination of measures for controlling sludge at the source.

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on
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AUTOMATED CONTROL OF SLUDGE TREAMENT SYSTEMS

May 20, 1980

Tokyo, Japan

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AUTOMATED CONTROL OF SLUDGE TREATMENT SYSTEMS

CONTENTS

Page

1. Introduction 91

2. Basic Flow Diagram of the Sludge Treatment System 93

3. Current Techniques of Operation and Control for Sludge Treatment Processes 96

3.1 Sludge Gravity Thickening 96
3.2 Sludge Digestion 97
3.3 Digested Sludge Elutriation 99
3.4 Chemical Conditioning 99
3.5 Sludge Dewatering 101
3.6 Sludge Incineration 102
3.7 Heat Treatment 104

4. Current Monitoring and Control Techniques for Sludge Treatment Systems 106

4.1 Type I 106
4.2 Type II 106
4.3 Type III ....

5. Future Progress in the Operation and Control of Sludge Treatment Processes 108

5.1 Sludge Thickening 108
5.2 Sludge Digestion 108
5.3 Digested Sludge Elutriation 109
5.4 Chemical Conditioning 109
5.5 Sludge Dewatering 109
5.6 Sludge Incineration 109

6. Future Progress in the Monitoring and Control of Sludge Treatment Systems 116

6.1 Type IV 116
6.2 Type V 116
6.3 Type VI 116

7. Discussion 117

1. INTRODUCTION

At present, various measuring instruments and control equipment are used in the operation and maintenance of sewage treatment plants, reflecting the remarkable development of electronic engineering in recent years. At several plants, operation and maintenance are carried out with digital computers.

In sludge treatment systems, which constitute a major part of sewage treatment plants, an increasing number of measuring instruments and control equipment is being applied.

These instruments and equipment are used in a manner to ensure stable operation of each sludge treatment system and to ensure high-quality effluent whatever the quantitive and qualitive fluctuations of sewage inflow to the plant. Thus, measuring instruments and control equipment are employed in each stage of the sludge treatment process, including the receiving of raw sludge from the sewage treatment system, the reduction of sludge volume, the stabilization of organic matter, and possible final disposal of the sludge.

Given that sludge from the sewage treatment system varies widely in both quality and quantity; for the prupose of receiving, reducing, stabilizing and disposing of the sludge, automated control of the sludge treatment system must be such that:

i) the maximum functioning of each stage can be obtained,

ii) the operation as the entire system can be optimized, and

iii) each constituent process can be functionally coordinated with the others.

The target performance of the automated control is as follows:

iv) improved sludge treatment efficiency of each process and at the same time of the system as a whole,

v) prevention of a temporary drop in sludge treatment capacity due to a partial failure of treatment facilities; namely, the prevention of stagnant accumulation of sludge in the sewage treatment system that may lead to deterioration of effluent quality,

vi) reduction of operation and maintenance costs of the sewage treatment plant, and

vii) improvement of working conditions for operators and workers engaged in sludge treatment.

2. BASIC FLOW DIAGRAM OF THE SLUDGE TREATMENT SYSTEM

Fig. 2.1 shows the sludge treatment systems of sewage treatment plants in Japan, classified by design sewage flow capacity.

Most sludge treatment systems in Japan are conbinations of either thickening-dewatering-incineration processes or thickening-anaerobic digestion-dewatering processes. In a limited number of small sewage treatment plants, the aerobic digestion process is employed instead of the anaerobic digestion process. Some large sludge treatment systems use a combination of thickening-heat treatment-dewatering-incineration processes.

In view of the present status and future prospects of sludge treatment systems, a sludge treatment system in which the processes shown in Fig. 2.2 are combined has been studied.

Sludge treatment systems consist of some or all of the processes A, B, C and D in Fig. 2.2. Processes A and B each have alternatives. Process A-(2) is a process of recent development. It uses either a centrifugal or flotation method for the mechanical thickening of excess sludge from the activated sludge process. Up until now, however, proper control methods for the mechanical thickening have not been reported.

In the alternative process to the process B, aerobic digestion is not employed, partly because its use is limited to small sewage treatment plants and partly because energy costs in Japan seem to make the development of aerobic digestion unlikely.

Each process shown in Fig. 2.2 has a sludge pump well or a storage tank. If each process is to be operated and controlled in an orderly way without being influenced by quantitative flucturations, and also to facilitate maintenance, periodic inspection and so on, it is desirable to install a storage tank of large capacity. Neverthless, no definite design concept has been established for determining the capacity of storage tanks.

Fig. 2.1 Classification of Sludge Treatment Systems

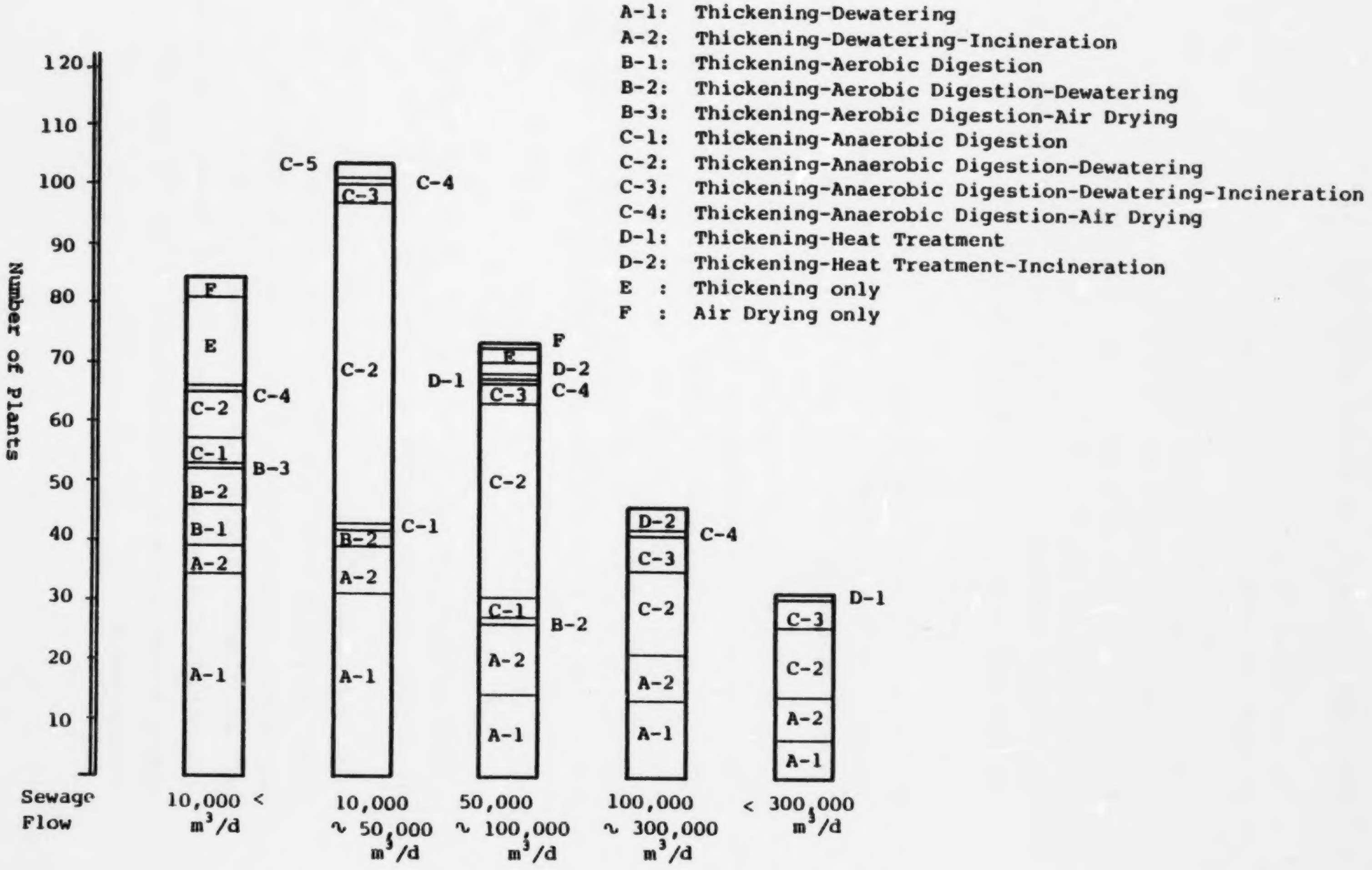
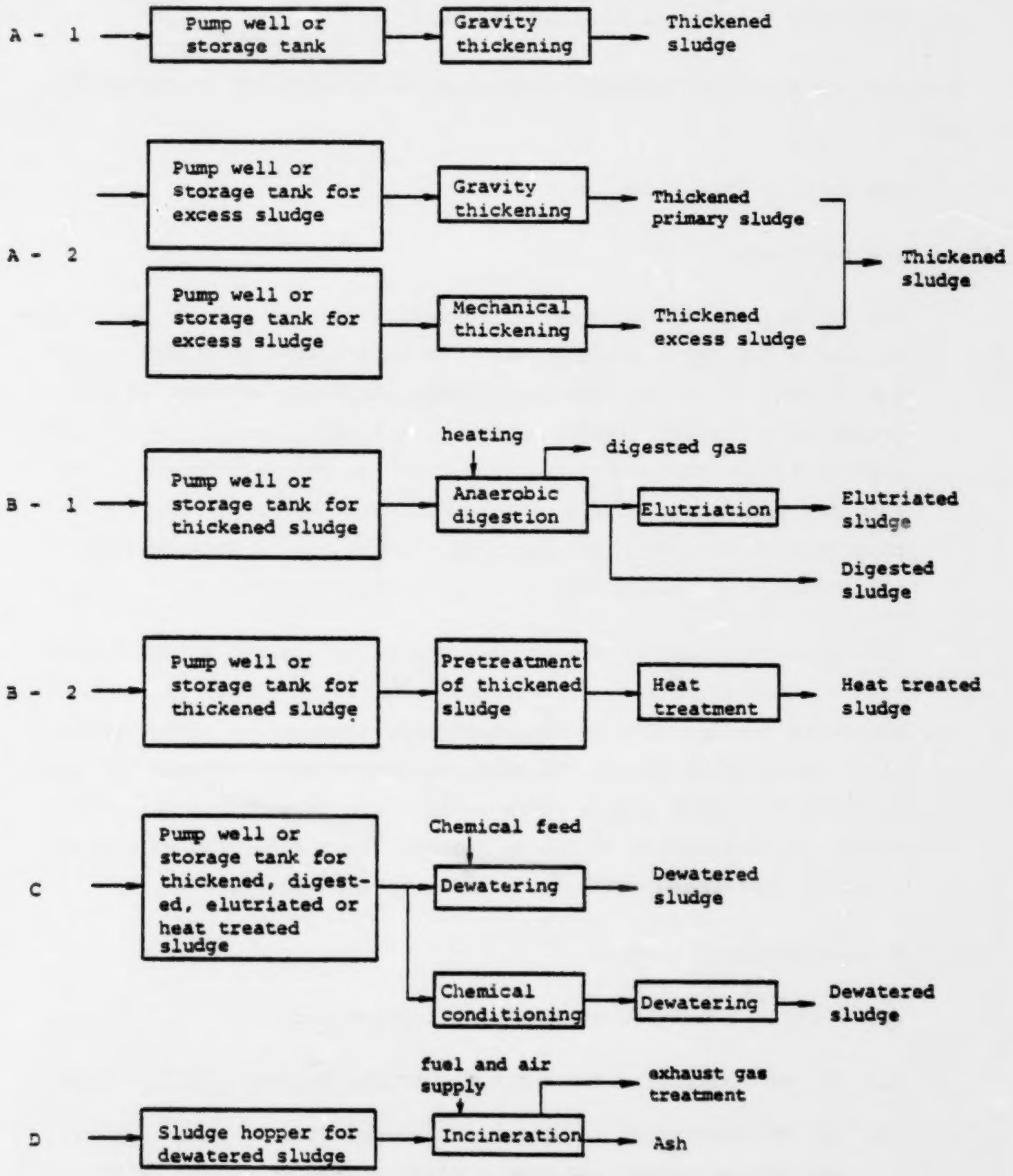


Fig. 2.2 Considered Flow Diagram of Sludge Treatment Processes


A - 1
Pump well or storage tank
Gravity thickening
Thickened sludge
A - 2
Pump well or storage tank for excess sludge
Gravity thickening
Thickened primary sludge
Pump well or storage tank for excess sludge
Mechanical thickening
Thickened excess sludge
Thickened sludge
B - 1
Pump well or storage tank for thickened sludge
heating
Anaerobic digestion
digested gas
Elutriation
Elutriated sludge
Digested sludge
B - 2
Pump well or storage tank for thickened sludge
Pretreatment of thickened sludge
Heat treatment
Heat treated sludge
C
Pump well or storage tank for thickened, digested, elutriated or heat treated sludge
Chemical feed
Dewatering
Dewatered sludge
Chemical conditioning
Dewatering
Dewatered sludge
D
Sludge hopper for dewatered sludge
fuel and air supply
Incineration
exhaust gas treatment
Ash

3. CURRENT TECHNIQUES OF OPERATION AND CONTROL FOR SLUDGE TREATMENT PROCESSES

Sludge treatment systems usually employ a sequence control system for interlocked operation of a number of instruments and equipment incorporated in its processes. Closed control loops are often used to regulate chemical feed for sludge dewatering, etc.

Process operation and control techniques now in wide use are described below.

3.1 Sludge Gravity Thickening

3.1.1 Inflow control

The sludge from the primary sedimentation tank is sent to the sludge thickener through a storage tank. In some sewage treatment plants, the drawing of sludge from the primary sedimantation tank is subjected to an ON/OFF control signal from a solid concentration meter monitoring the sludge drawing pipe, Excess sludge from the final settling tank continuously or intermittently returned at a fixed rate from the return-sludge storage tank to the primary sedimentation tank, or pumped directly to the sludge thickener.

In sewage treatment plants of relatively large size, if the sewage flow is almost the same as the design flow, the sludge is fed to the thickener continuously or intermittently because its sludge production amount is so great. In small sewage treatment plants, or even in large treatment plants where sewage flow is comparatively low, sludge is intermittently fed to the thickener according to a time schedule coordinated with the drawing of thickened sludge.

3.1.2 Thickened sludge control

There are four methods of drawing thickened sludge.

(1) ON/OFF control of the thickened sludge drawing pump by timer.

(2) ON/OFF control of the thickened sludge drawing pump by timer and preset counter to draw a fixed amount of sludge daily. Operation of the thickened sludge drawing pump is interlocked with the liquid level in the sludge digestion tank or sludge

storage tank to which it is connected.

(3) Sludge drawing control by sludge concentration.
The operation of the sludge drawing pump is started by a timer and stopped by an under-concentration signal from a solid concentration meter monitoring the sludge drawing pipe.

(4) Thickened sludge drawing control by the volume of solids.
In this method, thickened sludge drawing pump is started and stopped by signals from a solid concentration meter and preset counter so that the quantity of solids per drawing operation is limited to a preset value.

In comparatively large sewage treatment plants, when the sewage inflow is almost the same as the design flow, sludge drawing is carried out semi-continuously by method (1).

In small plants or in large plants that are treating a lower sewage flow than the design flow, sludge drawing is performed intermittently by method (2). There are few, if any, plants that employ methods (3) and (4). In most plants, solid concentration meters are used simply for indication and alarm purposes.

3.1.3 Sludge collector control

The sludge collector is run at a fixed speed, and is manually started and stopped.

3.2 Sludge Digestion

3.2.1 Sludge feed, drawing and transfer controls

Sludge feeding, sludge drawing and supernatant drawing are controlled only by a fixed delivery control process using a preset counter.

No other control processes are currently employed. Sludge feed control is interlocked with the thickened sludge drawing pump for the sludge thickener.

Transportation of sludge to the secondary digestion tank by gravity-feed, and the drawing of digested sludge is controlled by the level in the digested sludge storage tank or by a timer.

The supernatant from the secondary digestion tank is gravity-drawn. Seed sludge is transferred by the seed sludge transfer pump which is operated intermittently by a timer or manually. A level alarm is installed to protect the sludge digestion tank from a nagative pressure.

3.2.2 Agitator control

The sludge in the sludge digestion tank is mixed by gas agitation or by a mechanical agitation device. Gas agitation is generally used, however. Agitation is carried out either continuously or intermittently by a timer. Even intermittent agitation is carried out interlocked with the sludge feed pump or boiler when thickened sludge is being fed or the boiler is being operated.

In the secondary digestion tanks, in which scum is produced, agitation is carried out intermittently to break up the scum. There are two types of agitating diffusers for sludge mixing and scum breaking in secondary digestion tanks, and gas distribution control is also used for this purpose.

3.2.3 Temperature control

There are two ways of heating in sludge digestion tanks; one in which heat exchangers are used, and the other in which steam is blown in directly. In the former type, the sludge temperature is controlled by regulating the flow rate and temperature of the hot water. In the latter type, the temperature control is carried out by regulating the steam flow. Namely, the temperature of the sludge digestion tank is controlled to a set value in a feedback control process in which the hot water circulating pump or steam control value is subject to ON/OFF control.

Boilers must comply with the "Boiler and Pressure Vessel Safety Codes". Start-up must be carried out attended by an operator even if a remote control system is employed. The boiler is started and stopped based on sludge digestion tank temperature indication and alarm signals.

3.3 Digested Sludge Elutriation

3.3.1 wash water control

The wash water is fed in a fixed ratio to digested sludge volume. At present, however, strict ratio control is not performed, but instead a wash water valve or wash water pump is simply ON/OFF controlled in interlock with the digested sludge feed valve.

3.3.2 Elutriated sludge drawing control

The drawing of elutriated sludge is controlled by ON/OFF control signals to a sludge pump based on a sludge storage tank level signal or a timer signal.

3.4 Chemical Conditioning

Chemical conditioning facilities vary widely depending on what type of dewatering device is used and whether a lime and ferric chloride combination or polymer is used.

3.4.1 Chemical dissolving control

(1) Slaked lime

Slaked lime stored in a hopper or a silo is passed to a dissolving tank by a flow conveyor. It is then adjusted to milk of lime at 15% to 20% concentration. The dissolving is carried out continuously or in a batch process.

In the batch process, a contant lime feeder and a diluting water control valve are intermittently operated by a dissolving tank level signal and a timer signal in order to feed lime and water in a fixed ratio, mix and agitate them.

Chengeover from one dissolving tank to another is carried out automatically.

In the continuous process, lime and diluting water are supplied continuously in a fixed ratio. The agitator in the tank is automatically operated with a tank level signal.

The hopper sliding device, air blower, dust collecting fan, bag filter and other auxiliaries are operated in interlock with the constant feeder.

(2) Ferric chloride

Commercial ferric chloride has a concentration of about 38%. It is stored in a storage tank, and gravity-fed or transger pumped into a dissolving tank where it is diluted to about one quarter of its original concentration. Feeding is controlled by a dissolving tank level signal.

(3) Polymer

polymer is commerically available as a liquid or powder. The liquid polymer is about 10% in concentration and is stored in the storage tank. The polymer is diluted to about 1% in a mixer halfway to the dissolving tank. It is further diluted to between 0.3 and 0.5% in the dissolving tank.
Polymer powder is handled just the same way as slaked lime.

3.4.2 Chemical feed control

Chemical feed control is conducted either in a batch process or in a continuous process.

In the batch process, the running time of chemical feed pumps or chemical feed valves and of sludge feed pumps is controlled by timer signals and level signals in order to regulate the chemical feed ratios to preset values. In the continuous process, control valves or fixed delivery pumps are used to control the chemical feeds.

Chemical feed control by control valves is carried out just as for ordinary valves. Chemical feed control by fixed delivery pumps is carried out by stroke control and/or speed control. Chemical feed rates can be calculated from the stroke and speed of the fixed delivery pump, but usually is measured with a magnetic flowmeter. Chemical feed control is classified into two types; constant proportional ratio control in which a chemical is fed at a fixed ratio to the sludge flow, and mass constant proportional ratio control in which sludge flow and solid concentration are factors.

At present, constant proportional ratio control is most widely used. In either case, the optimum chemical feed rate for dewatering sludge is determined after a leaf test or examination of moisture content of the dewatered cake produced from the dewatering device.

3.4.3 Sludge feed control

If the sludge inflow is continuous, the chemical feed is carried out by constant ratio control. Thus, sludge feed should preferably be fixed. If a mixing tank is small, the level in the mixing tank will vary over a wide range, requiring the sludge feed pump to be started and stopped frequently. For this reason, the level in the mixing tank is regulated at a constant value by speed control of the sludge feed pump.

For this, control is carried out using the level in the mixing tank in the primary control loop as a control variable. The manipulating input of the sludge feed pump in the secondary loop is given by cascade control. This method is also applicable to the sludge-feed joint-mixing tank for a number of filter presses operated in parallel.

3.5 Sludge Dewatering

3.5 1 Vacuum filter

If vacuum filters are to maintain their rated performance at all times, it is necessary to always keep the sludge level in the vat at a constant value. The opening of the sludge feed valve in the sludge feed pipe from the mixing tank may be controlled automatically to keep the sludge level within the permissible range. With this method, however, the sludge feed pipe is likely to become clogged with solids. At present, sludge feed valves are operated simply by ON/OFF control in order to maintain sludge levels in vat.

Meandering of the filter cloth is detected by a sensing switch, which actuates an air cylinder to correct the meandering. If the air cylinder fails to correct the meandering, a safety switch functions to automatically shut down the rotation of the vacuum filter. Auxiliaries are operated in interlock with ON/OFF control of the vacuum pump.

3.5.2 Filter press

Filter presses are operated to a batch process in which the steps are:

pressure feeding sludge, compression of sludge, (air blowing), opening of frames, (filter cloth runing), filter cloth washing, and closing of frames. Operation is essentially of the sequence timer control type.

If the sludge feed pressure exceeds a preset value, the relif valve functions to shut down the sludge feed. The cycle time is set after examination of the moisture content of dewatered cake and filtering rate of chemical conditioned sludge.

3.5.3 Centrifuge

The speed of the sludge feed pump to the centrifuge is controlled to maintain the sludge feed rate at a constant value, and the concentration of solids is measured for mass calculations. Polymer feed is controlled to a constant ratio according to the results of mass calculations.
The sludge feed rate and chemical feed rate are determined according to solid recovery rate which is estimated from the state of the filtrate.

3.5 4 Belt press type dewatering device

The sludge feed rate is to be kept constant just as for centrifuging. The type and feed rate of polymer are determined bv the results of preliminary tests. And, the sludge feed rate, filter cloth tension, and filter cloth transfer speed are determined according to the thickness and moisture content of sludge cake obtained in actual operation. The meandering of the filter cloth is corrected with a band meandering adjustment meter.

3.6 Sludge Incineration

There are various types of incinerators including a multi-hearth furnace, fluidized bed furnace and rotary kiln. Furnaces of the same type vary in flow scheme, instrumentation and equipment, from manufacturer to manufacturer.

Control is divided into dewatered cake receiving and feed, fuel supply, combustion air supply and exhaust gas treatment subprocesses.

3.6.1 Dewatered cake receiving and feed

If the dewatering device and incinerator are to be operated concurrently, a sludge cake hopper is only necessary while the incinerator is out of services. On the other hand, if the dewatering device is run for 8 hours, while the incinerator is operated for 24 hours a day, a cake storage hopper is required.
To control the reciving of sludge cake at the feed hopper, a signal from a weigher or level switch in the cake hopper is used to start and stop the feed conveyor. Changeover to the storage hopper of the dewatered cake is carried out by means of a reversible feed conveyor. So that dewatered cake can be fed to meet the rated capacity of the incinerator, the weigher at one end of the feed conveyor issues a signal to control the speed of the constant feeder and the feed conveyor. This method has the drawback that if the moisture content of dewatered cake is high, it is impossible to feed the required amount of dewatered cake.

3.6.2 Fuel supply

To transfer heavy oil from an underground fuel tank to a service tank, the delivery rate of the fuel transfer pump is controlled by the levels in the respective tanks. Fuel is supplied from the service tank to the burner by a service pump, which is cascade controlled based on the furnace temperature.

3.6.3 Combusion air supply

Starting and stopping of the incinerator are under sequence control. For starting-up and stopping, the burners are controlled according to heating and cooling programs. In some cases, a holding program is used to the incinerator temperature from an arbitrary value to a specified holding value.

The combustion air supply is under ratio control with respect to the heavy-oil feed rate. For the multi-hearth furnace and other furnaces, the air-to-fuel ratio is set mechanically, and both air and fuel valves are controlled from a single station.

In some furnaces, a damper is used to control the furnace temperature. In some cases, the secondary air supply is controlled by a damper so as to keep the oxygen concentration in the exhaust gas at a fixed level.

The pressure in the multi-hearth furnace is kept negative by a blower. The negative pressure is kept constant by damper control.

3.6.4 Exhaust gas treatment

The exhaust gas is scrubbed through acid and/or alkaline solution. When the pH value of the scrubbing solution shifts outside a set span, a feed valve opens to make up the solution with a concentrated liquid. The scrubbed wastewater is neutralized as the same way. Electric dust precipitators, blowers, etc. are under sequence control.

3.7 Heat Treatment

3.7.1 Pretreatment facility

A drum screen, grit removal cyclone, etc. are provided as pretreatment facilities for the heat treatment process. These devices are program controlled in coordination with the operation of the thickened-sludge drawing pump.

3.7.2 Reaction facility

Raw sludge run out of a disintegrator is passed by a high-pressure sludge pump to a reactor via a heat exchanger. Water is passed through the reactor until the inside of reactor attains a specified temperature. When the inside of reactor has reached a specified temperature, sludge is changed in instead of water. The disintegrator and high-pressure sludge pump are operated for conditional control. The reactor is controlled so that the liquid level, temperature and pressure are held at their set values.

The heat-treated sludge is discharged from the reactor by a sludge discharge valve which is opened and closed intermittently by reactor liquid level and timer signals. So that the sludge is maintained at the reaction temperature, the exhaust gas vent valve is controlled with the opening of the steam feed valve at a preset steam feed rate.

The gas is vented intermittently by a timer. The flow rate is controlled by making use of an orifice flowmeter. The pressure of steam supplied from the boiler is controlled to a fixed value by a pressure regulator.

• • •

4. CURRENT MONITORING AND CONTROL TECHNIQUES FOR SLUDGE TREATMENT SYSTEMS

The monitoring and controlling of sludge treatment systems have been undertaken as part of the overall monitoring and control of entire sewage treatment plants.

This chapter discusses the monitoring and control of entire sewage treatment plants, not just matters concerned with sludge treatment systems. Monitoring and control systems have been changing from the hitherto hardware-related technology, as represented by analog controllers and relay boards, to highly flexible software-intensive technology relying on microcomputers and programmable sequences. Signal transmission systems have also been changing from direct transmission types using a network of cables to multiplex transmission types which require fewer cables.

The monitoring and control systems now in use in sewage treatment plants in Japan of three types as follows.

4.1 Type I

In the simplest type, monitoring and control is done with analog measuring instruments installed at the side of each treatment unit or on the control board in the sludge treatment building. The controllers are manually adjusted by an operator. The operator reads the analog measuring instruments and writes out a daily report.

4.2 Type II

This system is just the same as in Type I, but a computer is installed alongside and independent of the treatment facilities.
Data read by the operator from the analog measuring instruments on the control board is input to the computer. The information from the computer is indicated on a display panel, and at the same time is printed out as part of a daily report. The operator observes the display panel and the printed-out results makes a judgement, and manually adjusts the controller.

4.3 Type III

The monitoring and control methods are just as for Type I, except that the information from the measuring instruments located at each treatment device or facility is fed on-line to the computer. The information processed through the computer is indicated on a display panel and at the same time printed out as a daily report. The main memory of the computer is usually 16 ∿ 32 KW.

The operator evaluates the data on the display and printed-out results, and manually adjusts the controllers.

5. FUTURE PROGRESS IN THE OPERATION AND CONTROL OF SLUDGE TREATMENT PROCESSES

In the future, various instruments with higher reliability and greater durability will be developed, and the dynamic characteristics of sludge and its treatment processes will be understood more clearly. It will, then, be possible to introduce a computer-aided optimum process control system. Examples of operation and control systems for various treatment processes which will be a reality in the future and described below.

5.1 Sludge Thickening

(1) For the gravity thickener, the sludge drawing pump will be started and stopped on a signal from a sludge surface layer detector. At present, this method is being tried on an experimental basis.

(2) For the centrifugal thickener, the viscosity of thickened sludge will be measured to produce a signal for control of the differential speed of the outer and inner drums of the centrifuge to control the thickened sludge concentration to a fixed value. (See Fig. 5.1)

(3) For the floatation thickener, the turbidity of the supernatant will be measured to produce a signal to control the sludge feed rate.

5.2 Sludge Digestion

The concentration and pH of the thickened sludge to be fed into the sludge digesting tank will be measured, and alkaline chemical feed, sludge feed rate and seeding sludge recirculation rate will be under feedback control, proportional feedback control and/or feed-forward control.

This method is now being tested experimentally. Also, by measuring pH, temperature, and chemical composition and the amount of digested gas, the alkaline feed rate, seed sludge recirculation rate and heating temperature, etc. will be integrally controlled. (See Fig. 5.2)

5.3 Digested Sludge Elutriation

The alkalinity of the anaerobic digested sludge will be measured for ratio control of the wash-water feed rate. (See Fig. 5.3)

5.4 Chemical Conditioning

Measuring the moisture content of dewatered cake, turbidity of filtrate, pH of conditioned sludge, etc., the chemical feed will be under feedback control. A method in which pH of chemical conditioned sludge is controlled to a fixed value by measuring the pH of the filtrate discharged from the vacuum filter is now being tried on an experimental basis. (See Fig. 5.4)

5.5 Sludge Dewatering

(1) For the vacuum filter, the drum speed and agitator speed will be put under feedback control by measuring the moisture content of dewatered cake and cake thickness.

(2) For the filter press, the volume of iltrate will be measured for controlling the moisture content of dewatered cake to a fixed value. (See Fig. 5.5)

(3) For the centrifuge, sludge concentration and filtrate turbidity will be measured for controlling the sludge feed rate. If it becomes possible to measure the moisture content of dewatered cake quickly, it will become practical to control the differential speed between the outer and inner drums of the centrifuge.

(4) For the belt-press dewatering device, the control of filter cloth transfer speed may be practical if it becomes possible to measure the moisture content of dewatered cake quickly.

5.6 Sludge Incineration

The feed rate of sludge to the incinerator will be put under cascade control by measuring the furnace gas temperature. (See Fig. 5.6)
By measuring NOx and SOx in the exhaust gas, the operation of exhaust gas treatment equipments will also be controlled.

Fig. 5.1 Automated Control of Cetrifugal Thickener

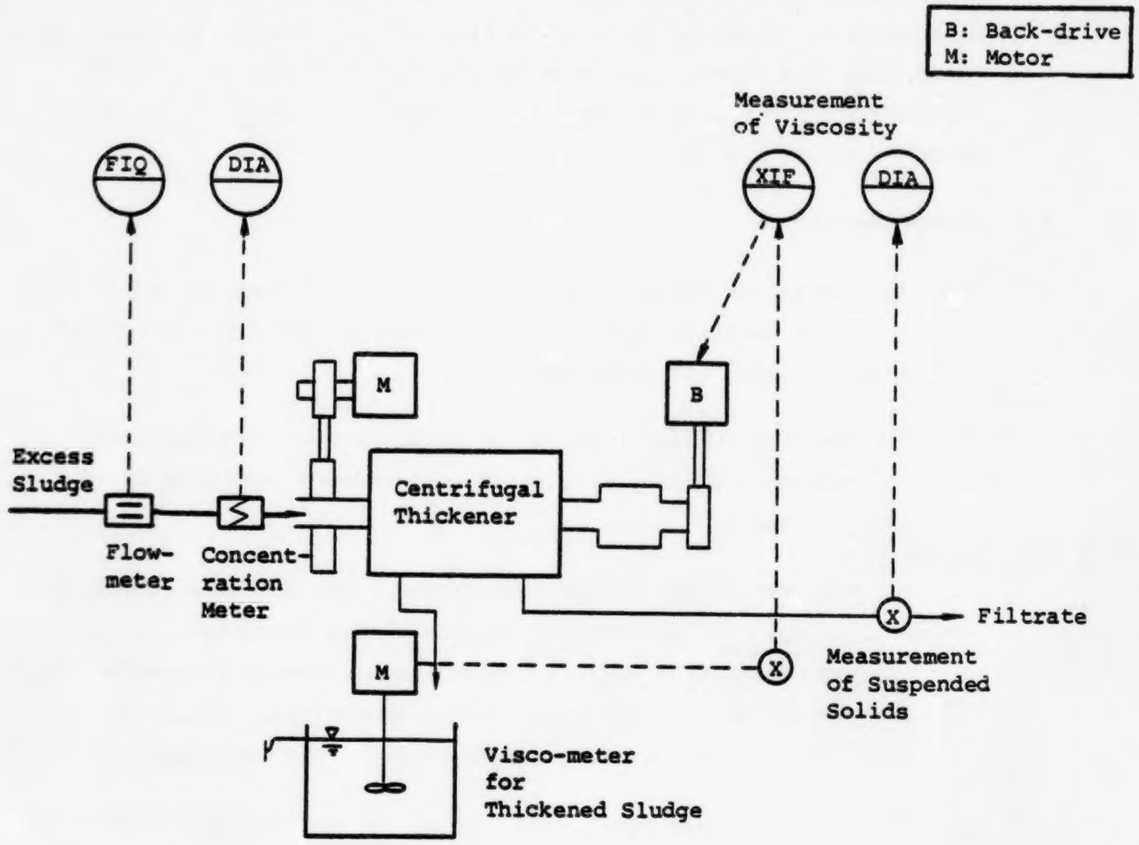

B: Back-drive
M: Motor
Measurement of Viscosity
FIQ
DIA
XIF
DIA
M
B
Excess Sludge
Centrifugal Thickener
Flow-meter
Concentration Meter
Filtrate
M
X
X
Measurement of Suspended Solids
Visco-meter for Thickened Sludge

Fig. 5.2 Automated Control of Digestion Tank by Using the Means of Measurements of Temperature, pH and Anaerobic Gas Composition

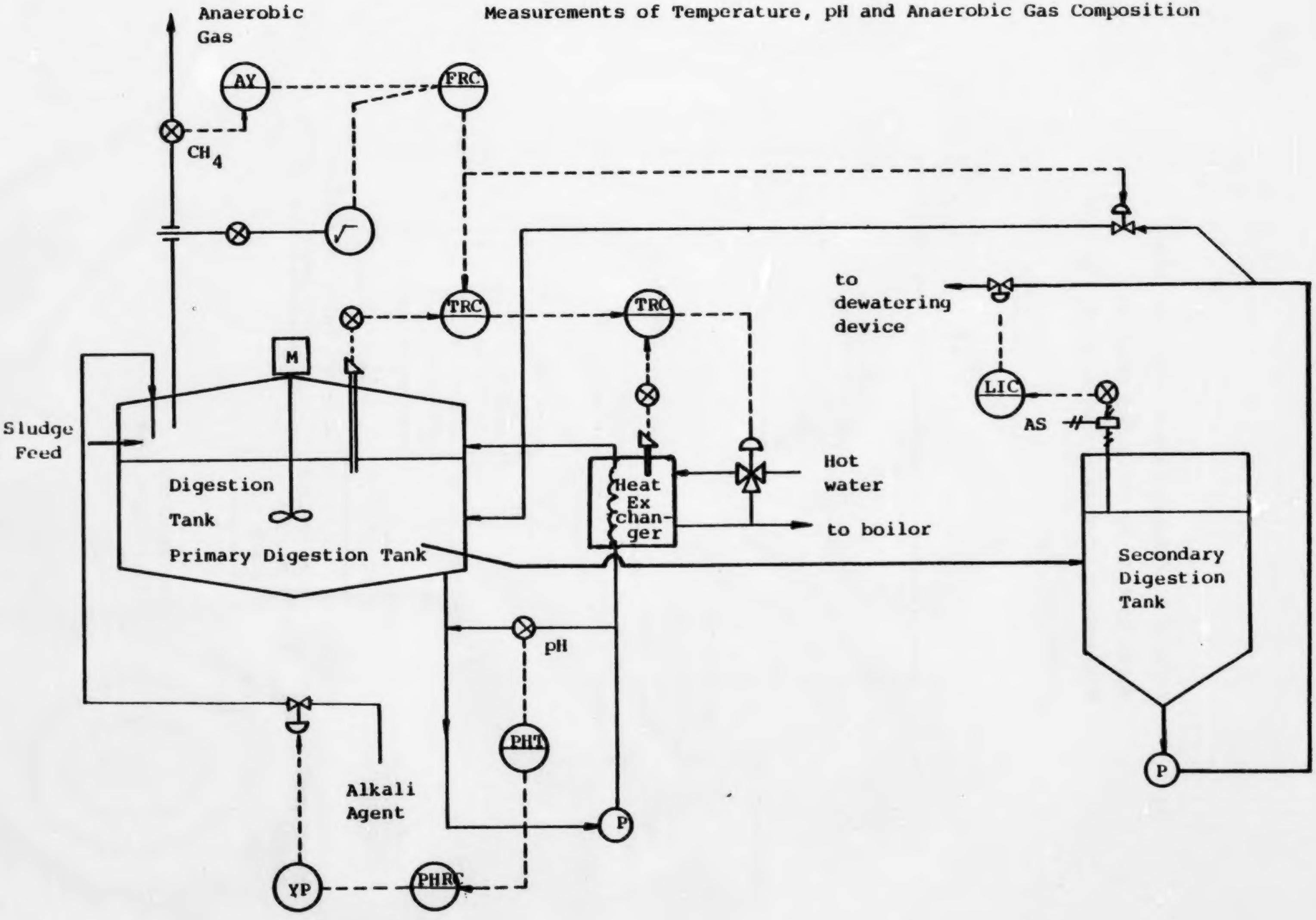
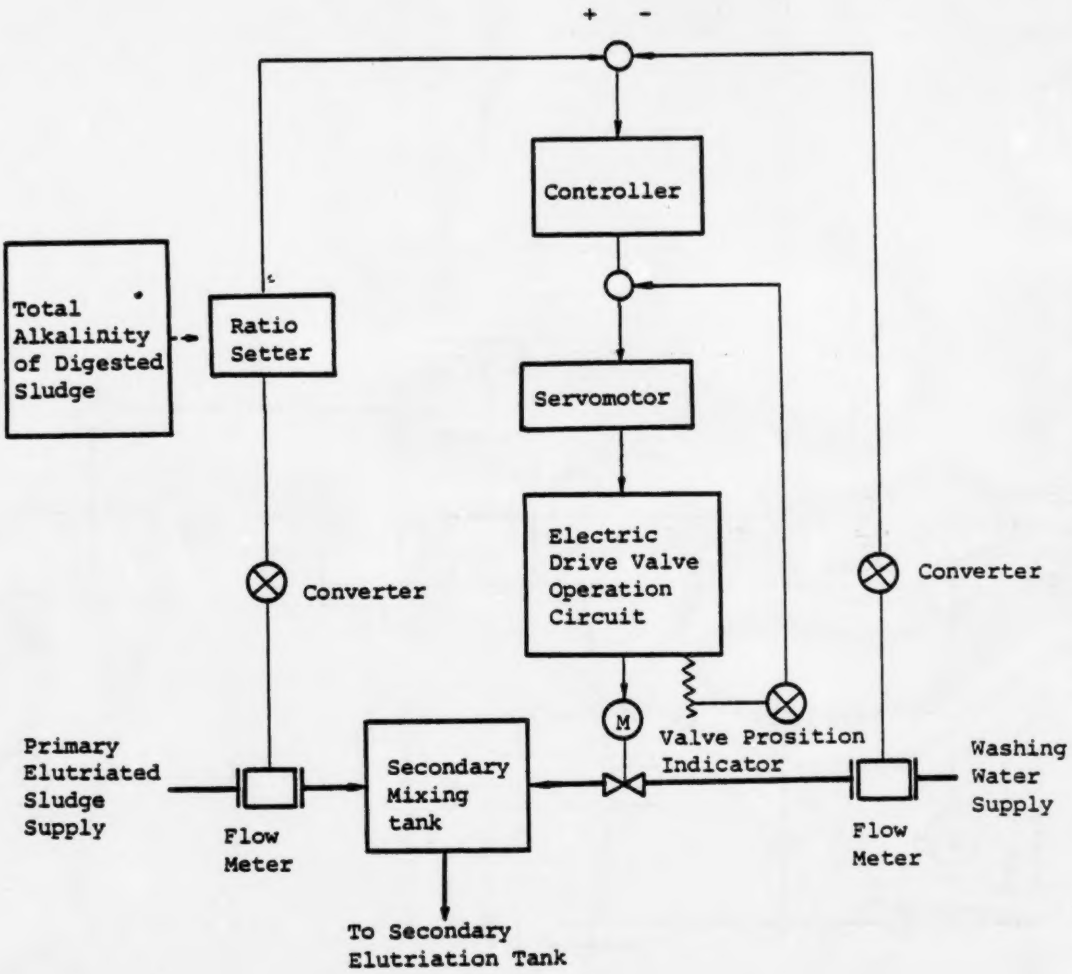

Anaerobic Gas
AY
FRC
CH4
TRC
TRC
M
to dewatering device
LIC
AS
Sludge Feed
Digestion Tank
Primary Digestion Tank
Heat Exchanger
Hot water
to boilor
Secondary Digestion Tank
pH
PHT
Alkali Agent
P
P
YP
PHRC

Fig. 5.3 Primary Elutriated Sludge - Washing Water Ratio Control by Using the Means of Measurement of Total Alkalinity in Digested Sludge



+
-
Controller
Total Alkalinity of Digested Sludge
Ratio Setter
Servomotor
Electric Drive Valve Operation Circuit
Converter
Converter
M
Valve Prosition Indicator
Primary Elutriated Sludge Supply
Flow Meter
Secondary Mixing tank
Washing Water Supply
Flow Meter
To Secondary Elutriation Tank

Fig. 5.4 Constant pH Control of Coagulant Dosing Sludge Fed for Vacuum Filter

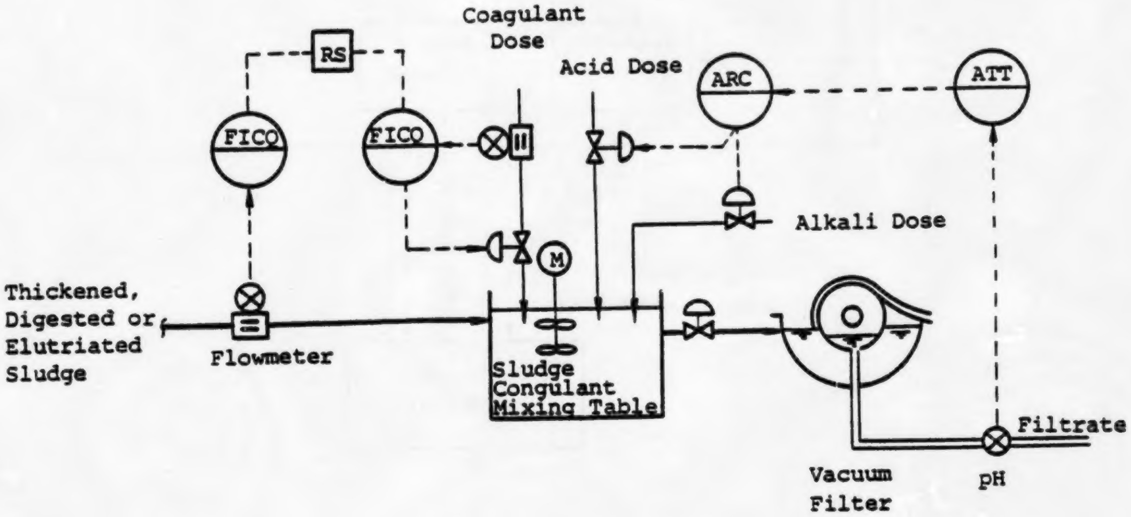

Coagulant Dose
Acid Dose
RS
FICQ
FICQ
ARC
ATT
M
Alkali Dose
Thickened, Digested or Elutriated Sludge
Flowmeter
Sludge Congulant Mixing Table
Filtrate
Vacuum Filter
pH

Fig. 5.5 Automated Constant Moisture Content Control for Filter Press

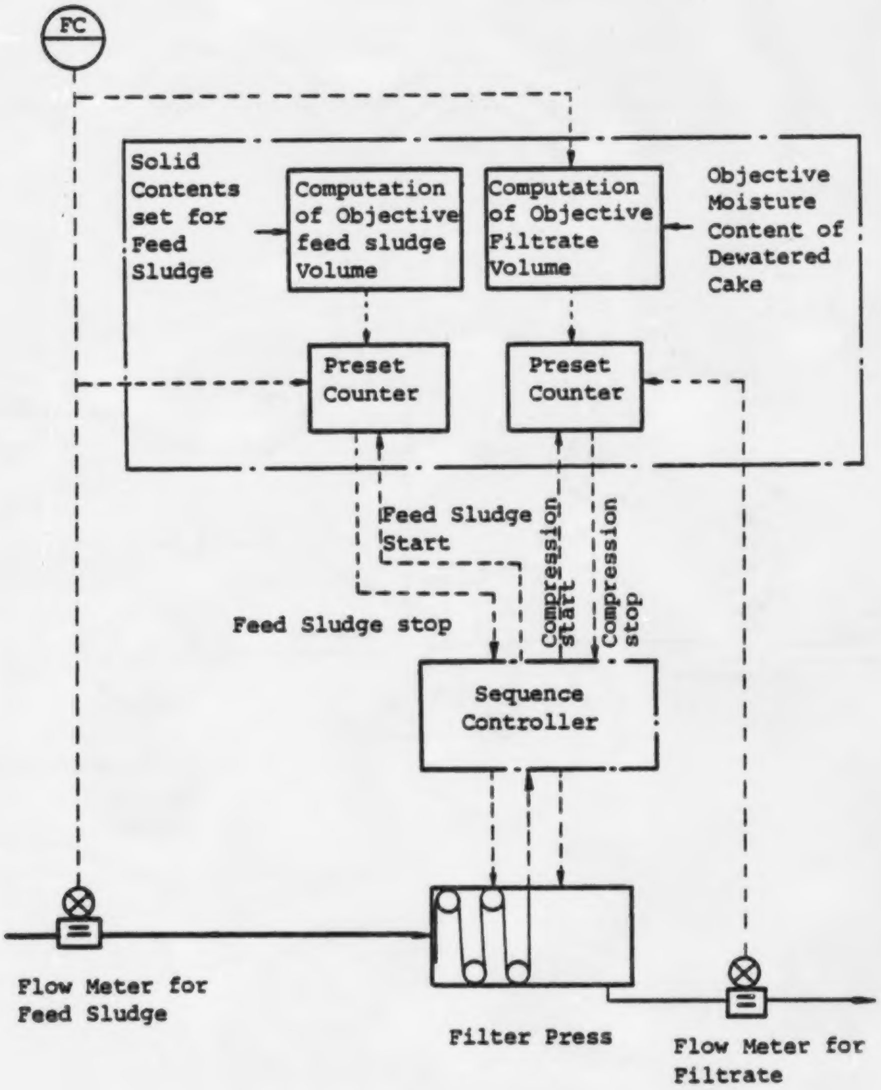
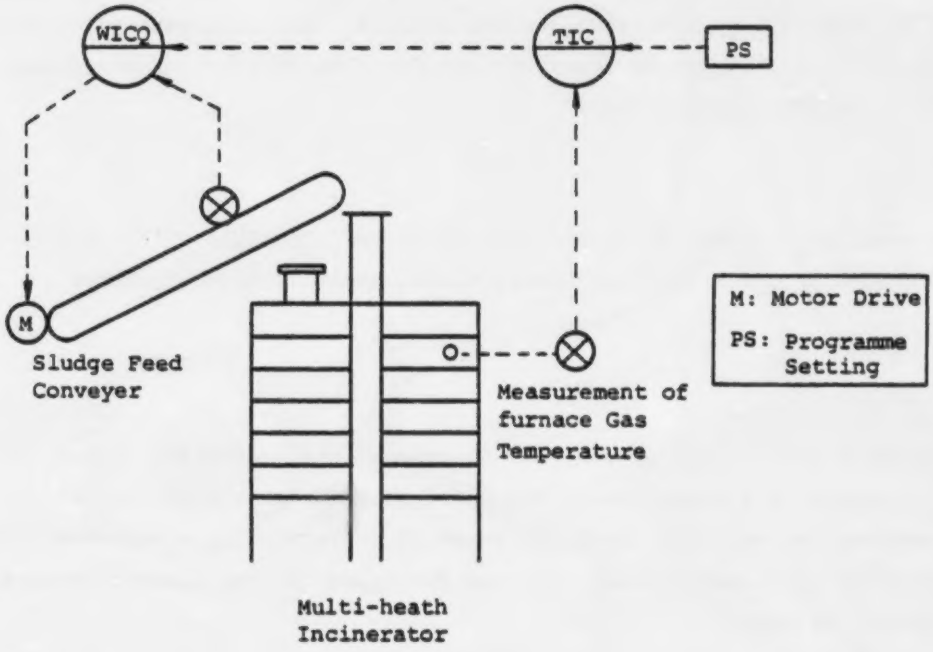

FC
Solid Contents set for Feed Sludge
Computation of Objective feed sludge Volume
Computation of Objective Filtrate Volume
Objective Moisture Content of Dewatered Cake
Preset Counter
Preset Counter
Feed Sludge Start
Compression start
Compression stop
Feed Sludge stop
Sequence Controller
Flow Meter for Feed Sludge
Filter Press
Flow Meter for Filtrate

Fig. 5.6 Dewatered Cake Control by Using the Means of Inner Gas Temperature Measurement for Multi-hearth Incinerator


WICO
TIC
PS
M
Sludge Feed Conveyer
Measurement of furnace Gas Temperature
M: Motor Drive
PS: Programme Setting
Multi-heath Incinerator

6. FUTURE PROGRESS IN THE MONITORING AND CONTROL OF SLUDGE TREATMENT SYSTEMS

Full development of measuring instruments and control equipment for the operation and control of all the processes will make it possible to computerize the automatic control of sludge treatment systems without difficulty. Control systems Types IV to VI below have already been tried experimentally in several sewage treatment plants in Japan.

6.1 Type VI

This type of control system conbines computer-aided automatic control with the Type III system described in Section 4.3 above. The controllers are set by computer outputs. Supervisory control is carried out at CRTs terminals at a central control station. The main memory capacity of the computers use is about 32 ∿ 64 KW. Usually, an auxiliary memory is required to main memory.

6.2 Type V

A computer is used for direct process control in place of the conventional analog controllers. Other functions are similar to those of Type IV.

6.3 Type VI

To use a wider range of computer functions, the controlled system is configured in several levels depending on the size and purpose of the component subsystems. Each subsystem is controlled by a separate computer with suitable capacity and functions to form a comprehensive control system.

Each process is under Direct Digital Computer control using a micro-computer, and the master computer is assigned a Set Point Control level function (including a data logging function). The memory capacity of the micro-computer is about 16 KW, and that of the master computer is about 32 ∿ 64 KW. Usually, an auxiliary memory is required.

7. DISCUSSION

Sludge treatment systems must always respond to fluctuations in the quantity and quality of the sludge produced in the sewage treatment system.

The sludge treatment system is a complex of various processes, and has several problems that must be solved for the purpose of ensuring stabilized performance of each process and of the system as a whole or a year long basis.

The treatment faciliteies or devices available for each sludge handing process have merits and demerits, and should be selected based on an in-depth study. In the planning stages of a sewage treatment plant, engineers are often unsure whether the treatment facilities or devices will be optimum or not for the sludge to be treated.

The treatment facilities or devices for these processes have been undergoing rapid technological innovations, which offers the risk of increasing investment in the selection of facilities or devices and formulation of sludge treatment systems from the long-range standpoint.

The sludge treatment systems require a large number of automatic measuring instruments for automation of each process or for the system as a whole.

Many quantitative measuring instruments which monitor sludge flow, level ect. have already been proved to work well through operation and maintenance in sewage treatment plants. On the other hand, such instruments as are used for measuring parameters like sludge concentration are still to be improved as they leave much to be desired in stability and reliability.

The measuring instruments for sludge treatment systems are always installed in severe environmental conditions than those for sewage treatment systems, and there are many problems awaiting solution in the operation and maintenance and also in the durability of the instruments

The operation and maintenance of automatic controlled sludge treatment systems needs well-trained operators, who must have a full understanding of the system configuration, the operating conditions of measuring instruments and control equipment, and also must always respond correctly and quickly to emergency situation.

At present, however, there are few, if any, such operators, and training provisions are not adequate. Yet, the automated control of sludge treatment systems will probably bring many benefits, as already discus It is therefore to be hoped that measuring instruments and control equipment are gradually upgraded.

The basic concepts at present for the sutomated control of sludge treatment system are as follows.

(1) Sludge treatment systems should be designed so that each process can be operated sutomatically. The treatment facilities and auxiliaries which are to be installed in batteries or trains should be designed to permit independent automatic control.
The engineering should be such that systems can continue service even if a few processes or treatment facilities becom faulty.

(2) Large-scale sludge treatment systems usually rely on a large number of treatment facilities and auxiliaries in a process.
Thus, it is necessary to duplicate these facilities and auxiliaries for alternate use.

(3) Sludge treatment systems should preferably be a combination of conventional site monitoring and control systems installed close to treatment facilities, and a centralized supervisory control system operated from a central control room station.
It is also desirable that data be collected at the central control station for future improvement of facilities or devices and to improve operation and maintenance activities.

(4) In large-scale sludge treatment systems, the data to be monitored controlled, measured and recorded becomes voluminous, and the analytical methods of collecting data in an efficient way should be studied in advance.

For sludge treatment systems of medium size or larger, data collection and recording should be carried out automatically.

For smaller sludge treatment systems, data collection and recording should be undertaken by operators, and analog or digital measuring instruments for data collection and recording should be installed wherever required.

(5) The present state-of-the-art measuring instruments leave much to be desired in adaptability, response and reproducibility.
So far as these instruments are concerned, the controllers associated with them should be designed to allow manual operation based on personnel judgement of actual treatment results. However, ample space should be allocated to accommodate future facilities for the automatic control which will become possible.

Seventh US/Japan Conference
on
Sewage Treatment Technology

ENGINEERING EVALUATION OF MUNICIPAL SLUDGE INCINERATORS

May 20, 1980

Tokyo, Japan

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Research and Technology Development Division,
Japan Sewage Works Agency

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CONTENTS

1. Introduction 123

2. Multi-Hearth Furnace 124

3. Fluidized Bed Furnace 127

4. Rotary Kiln 129

5. Comparison of Three Types of Incinerators from an Engineering Point of View 131

6. Economic Comparison of Dewatering - Incinerating facilities 132

   6.1 Preconditions 132

   6.2 Comparison of Utility Costs 134

   6.3 Comparison of Installation Costs and Occupancy Areas

   6.4 Summary of the Economic Comparison of Dewatering and Incineration 139

ANNEX-1 Calorific Value and Chemical Composition of Dewatered Cakes 141

ANNEX-2 Exhaust Gas Treatment Equipments 142

1. INTRODUCTION

Incineration of sewage sludge was started in 1965 in Japan. Installation of incinerators as of March 1978 is indicated in Table 1.1. As can be seen, 70 percent of the incinerators installed in the past are multi-hearth furnaces. Recently, however, the fluidized bed furnace and other new types have come to be employed.

However, with increasing installation of incinerators at sewage treatment plants in municipalities, some difficult problems have occurred. One particularly serious problems is air pollution from exhaust gas. Increasing operation and maintenance costs due to the sharp rise in prices of fuel is also a problem. Installation and operation of exhaust gas treatment facilities upon the demand of Pollution Control Departments of Prefectural Governments or inhabitants in the vicinity of sewage treatment plants are making the cost of incineration of sewage sludge much higher.

Many municipalities are finding it difficult to locate landfill sites for sewage sludge and have to reduce the volume of sludge as much as possible before dumping the sludge. In fact, some municipalities consider it unavoidable to incinerate the sludge, in spite of energy costs or environmental problems this right cause for the future.

In drafting this report, data was collected from detailed surveys of the operation of 3 multi-hearth incinerators, 2 fluidized bed incinerators and 2 rotary kilns. Data was also collected by questioning many engineers from Sewage Works Departments of municipalities who had experience in the design, operation and maintenance of incinerators. Based on this data, engineering and economic evaluations of these three types of incinerators were made.

These detailed surveys were conducted starting from the dewatered cake being fed into the incinerator and burnt until it was reduced to ash, dust, vapor and exhaust gas (including the discharge from the exhaust gas treatment).

During the survey period, the respective incinerators were so operated that there would be as little variation as practicable in the volume and characteristics of the feed cakes and heavy oil.

For the survey items, those that would permit a thorough understanding of the material and heat balances during incineration were chosen. The survey items thus include the volume and characteristics of feed cakes (moisture content; contents of volatile solids, carbon, hydrogen, nitrogen, sulfur, chlorides and oxygen; and upper and lower calorific values), the volume of air feed and its characteristics (temperature and humidity), the auxiliary fuel feed and its temperature, furnace temperature and exhaust gas volumes and temperatures at the furnace and heat exchanger outlets. The outlines of the survey result are shown in Table 1.2. For the rotary kilns, city gas of a low calorific value of 4,460 K cal/N·m^3 was used as an auxiliary fuel so that the feed of the auxiliary fuel is given after conversion to that of heavy oil in table 1.2. The overall air supply rate is expressed by the ratio of the air supplied and penetrated to the incinerator over the theoretical air required for combustion of the cakes and heavy oil.

Heat recovery from the exhaust gas from the incinerator is not made in the case of the multi-hearth incinerators because of low exhaust gas temperature. But in the case of the fluidized bed furnaces, 2~3 heat exchanger units were arranged in series for heat recovery between 880~900°C and 390~440°C. In the case of the rotary kilns, one unit had some heat recovery made, but the other had no heat recovery made at all.

2. MULTI-HEARTH FURNACE

The volume and characteristics of dewatered cakes fed to the multi-hearth furnace are subject to variation, although to a slight degree, in accordance with the conditions of sludge treatment, such as concentration and dewatering. But for such slight variation, stable operation is insured as the dry and burning zones in the furnace move up and down gently and automatically.

However, when dewatered cakes adapted for self-burning with a higher calorific value are fed into the furnace, they are burnt in the dry zone. For preventing such burning in the dry zone, it is necessary to introduce a large amount of air into the furnace and reduce the furnace temperature. Further, by the introduction of such a large amount of air, it is possible to prevent damage of the furnace wall, rotary arm and rotary cheese due to high temperature.

As a rule, the multi-hearth furnace is operated continuously. When operation is intermittent, burning of auxiliary fuel is required to preventing temperature change in the refractories of the furnace and for maintaining the furnace temperature.

According to measurement results, the dust contained in the exhaust gas at the outlet of the furnace is not much, about 1∿2 g per N.m^3 of exhaust gas. However, the odor in the exhaust gas at the furnace outlet was found to be very high, about 2,000∿8,700.

Fig. 2.1 illustrates a furnace of nominal capacity of 300 tons·wet base/day, and Fig. 2.2 and Table 2.1 show the results of trial calculations of the material balance and heat balance.

In designing a multi-hearth furnace, the points to be noted are given below.

1) Feed of the dewatered cakes to the furnace should be made at a constant rate and continuously.
2) Moisture content of the dewatered cakes is preferably 60∿80%.
3) When the moisture content of dewatered cakes is less than 55%, air should be fed in the dry zone of the furnace so that the furnace temperature does not rise abnormally.
4) Dewatered cake loading to the overall furnace area is adequate at 30∿40 kg·wet base/m^2·hr.
5) The temperature in the burning zone is preferably 700∿900°C to prevent the increase of combustibles contained in the ash and in consideration of the heat resistance of the furnace wall materials.
6) The temperature of the exhaust gas at the furnace outlet is preferably 250∿300°C.
7) The overall air supply rate to the furnace was actually 2.8∿2.9. But, to prevent the increase of combustibles in the ash and to reduce the volume of exhaust gas, it is preferably 1.4∿2.0.

8) For preventing the formation of clinker and protecting the rotary arm and cheese in the furnace, the auxiliary fuel burner should be of such a structure that the flame is not introduced into the furnace directly.
9) The concentration of dust emitted out of the chimney is reduced to less than 0.05 g/N.m^3, if an electric dust precipitator or an efficient water-scrubber is installed.
10) The concentration of SOx in the exhaust gas is reduced to less than 50 ppm, if water and/or chemical - absorption scrubbers are installed.
11) The concentration of NOx in the exhaust gas at the furnace outlet is 80∿120 ppm upon conversion with the oxygen concentration in air taken as 12%.
12) Noise abatement measures should be taken, if required. For example, noise generating machines such as induction fans, blowers, compressors, etc., can be installed in a sound absorbing chamber.

In operation and maintenance of the multi-hearth furnace, other points to be noted are as follows.

13) Operation of the furnace is made by maintaining the moisture content and feed of the dewatered cakes at a constant rate within the design capacity, controlling both the temperature of the dry zone adjacent to the burning zone automatically and the oxygen content in the exhaust gas at the furnace outlet adequately.
14) In the high temperature sub-zone in the furnace are contained mechanical rotary devices such as the mixing arm and cheese and several stages of extended furnace beds. In addition to the furnace wall, these are also subject to damage due to high temperature. Therefore, the furnace should be inspected periodically at least once a year (for such inspection, about 15 days is required).
15) The annual operating rate of the furnace is 85∿89%, including the time necessary for periodical inspection.

3. FLUIDIZED BED FURNACE

The volume and/or characteristics of the dewatered cakes fed to a furnace may vary more or less with time, depending on the dewatering condition. However, because of the large heat storing capacity of silica sand kept in the fluidized bed furnace, the furnace is insured against instable operation.

The fluidized bed furnace is characterized in that when it is operated intermittently, for example, if it is operated in the daytime only, it has a far smaller decrease in the inner temperature of the furnace, compared to the other type furnaces during the night time because of the great heat storage of silica sand. Thus, while the operation is discontinued, it is not required to keep the inner temperature of the furnace high with an auxiliary burner to protect the furnace wall. Further, in resuming operations, it does not take much time before the dewatered cakes are burnt steadily.

From measurement results, it was found that because of a high exhaust gas temperature of about 800°C at the furnace outlet, the odor concentration of the exhaust gas was relatively low at 300∿1700. But, the heat taken out by the exhaust gas is so much that installation of a heat exchanger is required. By such heat exchanger, the combustion air is heated to about 450°C before it is supplied to the furnace.

Fig. 3.1 shows an incinerator of a nominal capacity of 50 tons·wet base/day, and in Fig. 3.2 and Table 3.1 are shown the results of trial calculations of the material balance and heat balance of the incinerator, respectively.

In designing a fluidized bed furnace, the points to be noted are given below.

1) Feed of the dewatered cakes to the furnace should be made constantly and continuously.
2) According to the measurement result, the dewatered cake feed loading per unit area of the furnace was 200 kg/m^2·hr with cake having a moisture content of 80% and an ignition loss of 50% per dry solid, or 270 kg/m^2·hr with cake of a moisture

content of 65% and an ignition loss of 40% per dry solid. Further, the average velocity of the combustion wet gas in the furnace was about 0.8 m per second although it varied with the particle size and density of the fluidizing media.

3) The intake of dewatered cake to the furnace should be provided in the fluidizing layer or at a higher level of the furnace. One inlet is enough for furnaces of a nominal capacity of 50 tons·wet base/day or less.

4) The quiescent height of silica sand forming the fluidizing layer is generally 1∿1.5 m. About 1.5 times of the quiescent height is the height for fluidizing.

5) The free board level of the furnace must be about 3 times the fluidizing height of the silica sand in order to form an adequate fluidizing layer.

6) The proportion of radiant heat from the furnace wall to the input heat is about 5∿10%.

7) The overall air supply rate of the furnace was measured as 1.4∿1.6. But, to prevention increase of combustibles in the ash and to decrease the volume of exhaust gas, it is preferable to set an overall air supply rate at about 1.3 or less.

8) Combustion residues are all taken out of the furnace top so that preliminary dust precipitation equipment, such as a cyclone dust collector, is required immediately behind the furnace outlet.

9) The concentration of dust emitted from the chimney may be reduced to less than 0.05 $g/N \cdot m^3$, if an electric precipitator or an efficient water-scrubber is installed.

10) The SOx concentration in the exhaust gas is reduced to less than 50 ppm when water and/or chemical - absortion scrubbers are installed.

11) The NOx concentration in the exhaust gas at the furnace outlet is 90∿160 ppm upon conversion with an oxygen concentration taken as 12%.

12) Noise abatement measures should be taken, if required. For example, noise generating machines such as induction fans,

blowers, compressors, etc., can be installed in a sound absorbing chamber.

In the operation and maintenance of the fluidized bed furnace, other points to be noted are as follows.

13) Operation of the furnace is made by maintaining the moisture content and feed of the dewatered cake at a constant rate within the design capacity, and adequately controlling the temperature of the fluidizing layer and free board zone, furnace pressure and oxygen concentration at the furnace outlet.
14) Quantitative loss of the silica sand or fluidizing medium was, according to the measurement results, about 20 kg per ton of the dry solid of the dewatered cake fed. Supply of the silica sand depends on measuring the variation in pressure loss of the supplying air.
15) The furnace should be inspected periodically, at least once a year (about 7 days being required for such periodical inspection).
16) The annual operation rate of the furnace was 92%, including the time taken for periodical inspection.

4. ROTARY KILN

The rotary kiln is suited for stable operation as the dry and burning zones move forward or rearward automatically, according to fluctuations in the volume and characteristics of the dewatered cake fed to the kiln.

The rotary kiln is operated continuously as a rule, but when it is operated intermittently, burning of auxiliary fuel is required to prevent temperature change in the refractories of the furnace and for maintaining the furnace temperature.

For the rotary kiln, it is necessary to install an emergency combustion column drive in order to cope with any sudden electric power outage. This drive is intended to prevent deformation of the combustion column by discharging the cake remaining in the kiln.

With respect to the two rotary kilns shown in Fig. 4.1, the results of trial calculations on their material and heat balances are shown in Fig. 4.2 and Tables 4.1 and 4.2, respectively.

In designing a rotary kiln, points to be noted are given below.

1) Feed of the dewatered cakes to the kiln should be made constantly and continuously.
2) The heat capacity coefficient in the dry zone of the kiln is about 200 K cal/m^3·hr·°C with a combustion column diameter of 1.6 m.
3) The flow capacity of combustion wet gas per unit cross-sectional area of the combustion column is about 3,300 kg/m^2·hr.
4) The exhaust gas temperature at the outlet of the combustion column is preferably set at about 300°C.
5) The radiant calorific value of the column should be less than 15% of the input heat.
6) According to the measurement results, the overall air supply rate to the kiln was 2.4∿3.2. Depending on the structural characteristics, the air penetrating tends to become greater. It is desirable to reduce the air penetrating through the rubbing part of the combustion column so far as practicable so that an overall air supply rate of 1.4∿2.2 is provided.
7) The concentration of dust emitted out of the chimney may be reduced to less than 0.05 g/N·m^2, if an electric precipitator or an effective water-scrubber is installed.
8) The SOx concentration in the exhaust gas may be reduced to less than 50 ppm, if water and/or chemical - absorption scrubbers are installed.
9) The NOx concentration in the exhaust gas at the combustion column outlet is 70∿100 ppm upon conversion with an oxygen concentration set at 12%.
10) Noise abatement measures should be taken, if required. For example, noise generating machines such as induction fans, blowers, compressors, etc. can be installed in a sound absorptive chamber.

In operation and maintenance of the rotary kiln, other points to be noted are as follows.

11) Operation of the kiln should be made by maintaining the moisture content and feed of dewatered cakes at a constant rate within the design capacity and, at the same time, controlling the exhaust gas temperature and oxygen concentration at the combustion column outlet.

12) There are no mechanical rotary parts present in the combustion column, but the column itself rotates. Therefore, periodical inspection should be carefully carried out for missing refractories in the combustion column, wear on the rollers supporting the combustion column and adjustment of the rubbing part between the rotary combustion column and the fixed hood (about 7 days being required for the inspection).

13) The annual operation rate of the kiln was 80%, including the time required for periodical inspection.

5. COMPARISON OF THREE TYPES OF INCINERATORS FROM AN ENGINEERING POINT OF VIEW

Upon the results of these detailed surveys, the three types of incinerators were compared with one another from the engineering point of view. The results are as follows.

1) The multi-hearth furnace has the furnace bed fixed, and agitation and transportation of the cake in the furnace are made as the rake rotates. The rotary kiln has the cake in the kiln agitated and transported by rotating the combustion column itself. Accordingly, both furnaces are simple to operate.

   On the other hand, the fluidized bed furnace involves consumption of the silica sand due to its fluidizing and cracking so that technical skill is required during operation to maintain the fluidizing layer in optimum condition to burn the cake continuously.

2) Incinerators with design capacities of more than 100 tons·wet base/day per unit were limited to the multi-hearth furnace.

3) When operation is intermittent, the multi-hearth furnace and rotary kiln require burning of auxiliary fuel to maintain the temperature during suspension periods. In this respect, the fluidized bed furnace is better in that it can be started without auxiliary heating.

4) As regards the odor of exhaust gas at the furnace outlet, the fluidized bed furnace has the least.

5) The fluidized bed furnace allows operation with a lower overall air supply rate than the other furnaces.

6) For the fluidized bed furnace, it is necessary to install a cyclone dust collector to separate combustion residue or ash from exhaust gas.

7) With respect to the SOx concentration in the exhaust gas at the furnace outlet, these three furnaces are nearly the same.

8) For the NOx concentration in the exhaust gas at the furnace outlet, the three furnaces are nearly the same. The NOx concentration calculated with an oxygen concentration taken as 12% is lower than 250 ppm.

6. ECONOMIC COMPARISON OF DEWATERING - INCINERATING FACILITIES

6.1 Preconditions

With the sewage sludge generated from a sewage treatment facility having a design capacity of 50,000 m^3/day to 200,000 m^3/day dewatered by a vacuum filter, filter press or centrifuge, and the dewatered cake burnt in use of multi-hearth furnace, fluidized bed furnace or rotary kiln, the respective combinations at the facilities were compared with one another with respect to the utility cost, installation cost and occupancy area.

Calculations were based on data obtained from detailed surveys and other approved methods.

A. Sewage flow

50,000 m^3/day, 125,000 m^3/day and 200,000 m^3/day, respectively.

B. Sludge production and its characteristics

1) Dry solid generated in sewage

11.5 tons/day, 28.75 tons/day and 46.0 tons/day for the foregoing sewage flows respectively.

2) Moisture content of thickened sludge

96%.

3) Composition of dry solids in sludge

Volatile solids, 60%; ash, 40%.

4) Lower calorific value of volatile solids (According to ANNEX-1)

5,400 K cal/kg·VS.

5) Composition of volatile solids (According to ANNEX-1)

With vacuum filter and filter press: $(C_{100}H_{180}N_{11}O_{43})_7S$; and with centrifuge: $(C_{100}H_{180}N_{11}O_{43})_5S_7$.

C. Operation of dewatering devices

1) Operation hours

6 days per week, 6 hours per day.

2) Kinds of coagulants and dosing rate

With vacuum filter and filter press: lime 35% and ferric chloride 7% for dry solids (by weight); and with centrifuge: Polymer 1% for dry solids (by weight).

3) Dewatering rate

Vacuum filter: 15 kg·dry base per hour per m^2 of filter area

Filter press: 5 kg·dry base per hour per m^2 of filter area

4) Moisture content of dewatered cake

With vacuum filter: 77%.

With filter press: 65%.

With centrifuge: 80%.

5) Scale of dewatering devices used in calculations

Vacuum filter: 20 m^2, 30 m^2, 40 m^2 and 50 m^2, respectively, of filter area

Filter press: 50 m^2, 75 m^2, 100 m^2 and 125 m^2, respectively, of filter area

Centrifuge: 5 m^3, 10 m^3, 15 m^3 and 20 m^3, respectively, per hour.

D. Operation of incinerators

1) Operating hours

   7 days per week   24 hours per day

2) Auxiliary fuel

   A-type heavy oil (lower calorific value, 10,280 K cal/kg; and specific gravity, 0.89).

   Composition: C, 85.5%; H, 12.6%; O, 0.5%; N, 0.5%; and S, 0.9%.

3) Ambient conditions

   Temperature, 20°C; absolute humidity, 0.01 kg-H_2O/kg·air; and specific gravity, 1,293 kg/N·m^3.

4) Dewatered cake feed temperature: 20°C

5) Cooling water temperature: 20°C

6) Exhaust gas temperature at furnace outlet

   With multi-hearth furnace: 250°C
   With fluidized bed furnace: 800°C
   With rotary kiln: 500°C

7) Ash discharge temperature

   With multi-hearth furnace: 250°C
   With fluidized bed furnace: 800°C
   With rotary kiln: 250°C

8) Overall air supply rate

   With multi-hearth furnace: 2.9
   With fluidized bed furnace: 1.6
   With rotary kiln: 2.6

9) Furnace heat loss

   15% for each of the three types of furnace.

10) Supply of silica sand in the case of fluidized bed furnace

    20 kg per ton of dry cake.

11) Scale of incinerators used in calculation

    With multi-hearth furnace: 30 tons, 40 tons, 50 tons and 100 tons per day, respectively.
    With fluidized bed furnace: 30 tons, 40 tons, 50 tons and 100 tons per day, respectively.
    With rotary kiln: 30 tons, 40 tons, 50 tons and 100 tons per day, respectively.

E. Flow diagram of exhaust gas treatment equipment (Reference ANNEX-2)

With multi-hearth furnace:

$\xrightarrow{(250^\circ C)}$ Spray scrubber $\longrightarrow$ Chemical-absorption scrubber

$\xrightarrow{(40^\circ C)}$ Discharge.

With fluidized bed furnace:

$\xrightarrow{(800^\circ C)}$ Heat exchangers $\xrightarrow{(250^\circ C)}$ Dry cyclone dust collector

$\longrightarrow$ Bentury scrubber $\longrightarrow$ Chemical-absorption scrubber

$\xrightarrow{(40^\circ C)}$ Discharge.

With rotary kiln:

$\xrightarrow{(500^\circ C)}$ Heat exchanger $\xrightarrow{(250^\circ C)}$ Bentury scrubber $\longrightarrow$

Chemical-absorption scrubber $\xrightarrow{(40^\circ C)}$ Discharge.

F. Unit costs of materials

Auxiliary fuel (A-type heavy oil), ¥35/ℓ; Electric power, ¥10/KWH; Slaked lime, ¥30/kg; Ferric chloride, ¥25/kg in 38% solution; Polymer, ¥1,400/kg; Sodium hydroxide, ¥56/kg; Industrial water, ¥15/m^3; and Silica sand, ¥25/kg.

G. Occupancy area

It was assumed that both the dewatering equipment and incinerator would be arranged horizontally.

6.2 Comparison of Utility Costs

6.2.1 Calculations of the utility of dewatering devices

To obtain the utility of dewatering devices, the following estimate was made.

1) The dosed quantity of coagulant corresponding to the dry solid of the sludge generated in each case was calculated.
2) The supplied quantity of industrial water used in dewatering was calculated for cleaning the filter cloth, dissolving the coagulant, water sealing of the vacuum pump, etc.

3) Electric power consumption was calculated for the vacuum pump, oil-pressure pump, sludge press pump centrifuge, etc. For continuous operation machines, power was estimated as 0.8 of the value obtainable by multiplying the motor output by the hours of operation of dewatering devices. For machines of intermittent operation, it was taken as 0.4 of the value obtained by multiplying the motor output by the hours of operation of the dewatering devices. Table 6.1 shows the primary units of the utilities for dewatering devices.

6.2.2 Calculation of the heavy oil consumption for dewatered cake burning

In order to obtain the heavy oil consumption for dewatered cake burning, the following estimate was made.

1) The input heat to the furnace was calculated as the sum of the calorific value of dewatered cakes and that of heavy oil.
2) The potential heat of dewatered cake was calculated from the volume and moisture content of the cake fed, the dosed quantity of coagulants and lower calorific value of the dewatered cake. The lower calorific value of dewatered cakes is 5370 K cal/kg·VS, as shown in ANNEX-1.
3) The potential heat of heavy oil was calculated from the quantity of heavy oil required per hour and its lower calorific value.
4) The output heat from the furnace was calculated as the sum of the heat which the moisture in the dewatered cakes removes as exhaust gas, heat taken out upon discharge of ash, heat taken out as exhaust gas upon combustion of volatile materials and heavy oil, and heat loss from the furnace wall, etc.
5) Heavy oil consumption was calculated by expressing the input and output heats as calorific volumes per hour, and assuming that these were equal.

The amount of heavy oil consumed for combustion of dewatered cake by type of furnace and by moisture content of dewatered cake is shown in Fig. 6.1.

It will be seen from Fig. 6.1 that heavy oil consumption per unit weight of cake increases sharply with an increasing moisture content in dewatered cakes or a decreasing lower calorific value of dewatered cake. It will also be seen that when the same dewatered cake is burnt, the fluidized bed furnace consumes the largest amount of heavy oil. This is because the temperature of exhaust gas from the fluidized bed furnace is as high as 800°C and the potential heat of the combustion gas of volatile solid and heavy oil is remarkably high.

Plotting the results of the detailed surveys shown in Table 6.1 into Fig. 6.1, it can be seen that heavy oil consumption is approximately the same, except for the fluidized bed furnace G. This fluidized bed furnace G has an exhaust gas temperature of 880°C at the furnace outlet, or a greater rate of the potential heat of combustion gas being removed, thus producing a greater consumption of heavy oil than that of calculation shown in Fig. 6.1.

In the case of the multi-hearth furnace shown in Fig. 6.1 where the moisture content is 65% and the lower calorific value is 2750 K cal per kg, of dry cake, the dewatered cake is suited for self-burning.

6.2.3 Calculation of utilities, other than heavy oil required for combustion

In order to obtain values for the utilities, except heavy oil, required for combustion, the following estimation was made.

1) The amount of sodium hydroxide consumed was calculated from the weight of SO_2 in dewatered cake and the quantity of sulfur contained in heavy oil.
2) The amount of industrial water consumed in the scrubber was provided with reference to the result of detailed surveys.
3) Electric power consumption was calculated as stated in paragraph 6.2.1 with all of the motors of conveyor, induction fan, fluidizing blower, etc. classified into those used continuously and those used intermittently.

6.2.4 Utility cost for dewatering

The utility cost was calculated by multiplying the primary units shown in Table 6.1 by the unit costs. It is highest with the vacuum filter and lowest with the centrifuge, but there is not much different between devices, the vacuum filter being only 7% and the filter press by only 4% higher than the centrifuge.

The proportion of the cost of dosed coagulant in the utility cost is 90∿93% in the dewatering devices.

6.2.5 Utility costs for incinerating

Estimates of the utility costs for incineration are shown in Table 6.2.

The utility cost required for burning the same dewatered cake is lowest with the multi-hearth furnace and highest with the fluidized bed furnace.

6.2.6 Utility costs for dewatering and incinerating

In the utility costs of the combination of dewatering devices and incinerator shown in Table 6.2, the combination of filter press and multi-hearth furnace is the cheapest. In this case, the dewatered cake is self-burning and heavy oil is not required. Where the filter press is used, the utility costs are generally low. On the other hand, the highest utility cost results from the combination of vacuum filter and fluidized bed furnace, the utility cost being 5.5 times that of the combination of filter press and multi-hearth furnace.

Fig. 6.2 illustrates the utility cost required for dewatering and incineration per unit of sludge solid for each of the sewage flows of 50,000 m^3/day, 125,000 m^3/day and 200,000 m^3/day, respectively.

In each case of sludge production of the sewage flows, the combination of filter press and multi-hearth furnace is the cheapest. In this case, however, the dewatered cake is self-burning and it may become difficult to control the furnace. On the other hand, the highest utility cost is the combination of vacuum filter or centrifuge and fluidized bed furnace.

However, the multi-hearth furnace and rotary kiln may require after-burning of exhaust gas to reduce foul odors, depending on the location of the installation. Such after-burning of exhaust gas is not required in the fluidized bed furnace. Accordingly, depending on the installation site, the combination of filter press and fluidized bed furnace may have the cheapest utility cost.

6.3 Comparison of Installation Costs and Occupancy Areas

Indexes of the installation costs of the combinations of the 3 types of dewatering devices and incinerators are shown in Fig. 6.3. The proportions of dewatering devices and incinerators in the installation cost are shown in Table 6.2.

In Fig. 6.3, the installation cost was highest when the filter press was used for dewatering, regardless of the type of the incinerator. This is because the installation cost of the filter press is about 3 times higher than those of the dewatering devices.

The areas of occupancy of the combinations of the 3 types of dewatering devices and incinerators are shown in Fig. 6.4. Further, in Table 6.4 are shown the areas of occupancy of the dewatering devices and incinerators respectively.

The area of occupancy of the combination of dewatering devices and incinerators was smallest when the fluidized bed furnace was employed, regardless of the type of dewatering device.

6.4 Summary of the Economic Comparison of Dewatering and Incineration

1) In the comparison of the utilitiy costs for vacuum filter, filter press and centrifuge, the centrifuge is the cheapest. With the filter press and vacuum filter, the utility costs were higher by 4% and 7% over the centrifuge, erspectively.
2) The utility cost for incineration is cheapest with the multi-hearth furnace and most expensive with the fluidized bed furnace.
3) According to calculation results with exhaust gas treatment made by a scrubber and a chemical absorption scrubber, the fluidized bed furnace was highest in the consumption of heavy oil. However,

if after-burning of the exhaust gas is undertaken, in the case of multi-hearth furnace or rotary kiln, the foregoing relationship will be reversed.

4) The utility cost of the combination of dewatering devices and incinerators is cheaper when the filter press is employed as a dewatering device. Particularly, in the case of the combination of filter press and multi-hearth furnace, the utility cost is cheapest. On the other hand, the combination giving the highest utility cost is that of vacuum filter and fluidized bed furnace. The cost is as high as 5.5 times that of the combination of filter press and multi-hearth furnace.

5) The installation cost according to the combination of dewatering devices and incinerators is highest when the filter press is employed as a dewatering device regardless of the type of incinerator.

6) The area of occupancy of the combination of dewatering devices and incinerators is smaller when the fluidized bed furnace is employed, regardless of the type of dewatering device.

ANNEX-1 CALORIFIC VALUE AND CHEMICAL COMPOSITION OF DEWATERED CAKES

Dewatered cakes collected from 20 sewage treatment plants were subjected to chemical analysis. The results are shown in Table A.1.

The contents of carbon, hydrogen and nitrogen per unit weight of the volatile solid did not differ significantly from cake to cake. But, it has been found that the content of combustible sulfur differs greatly from cake to cake. This is, with the dewatered cakes having polymer or no coagulant, combustible sulfur accounts for about 90% of the content of total sulfur, but in the case of the dewatered cakes having both lime and ferric chloride, it is only 3∿10%. In the latter case, it is supposed that a greater part of sulfur contained in the dewatered cake will remain in the ash upon burning.

If it is assumed upon the result of elementary analysis that the volatile solid gasifying at burning entirely consists of oxygen, except the foregoing four elements, the following molecular composition is conceivable.

- Volatile solid contained in dewatered cake having both lime and ferric chloride.
  $= (C_{100} H_{180} N_{11} O_{43})_7 S$ .... (A.1)
- Volatile solid contained in dewatered cake having polymer or no coagulant.
  $= (C_{100} H_{180} N_{11} O_{43})_5 S_7$ .... (A.2)

Then, assuming that upon complete combustion of such volatile solids, carbon would be converted to CO_2, hydrogen to H_2O, sulfur to SO_2 and nitrogen to NO for 2% by weight, the remaining being emitted as nitrogen gas, calculations were made on the theoretical quantities of air and combustion gas. The results are as shown in Table A.2.

Next, the higher calorific value of measurement (Qh) is given as a function of the ratio of the weight of volatile solids to the weight of dry cake (V), as shown in Fig. A.1.

That is,

$$Qh = 58V \quad \ldots\ldots\ldots\ldots \quad \text{(A.3)}$$

Regardless of the kind of dewatered cake, the higher calorific value per kg, of volatile solid is 5,800 K cal.

The lower calorific value per kg, of volatile solid is given, from formulas (A.1), (A.2) and (A. 3) as 5,370 K cal.

ANNEX-2 EXHAUST GAS TREATMENT EQUIPMENTS

With the multi-hearth furnace and rotary kiln, the dust concentrations at the furnace outlet were 1∿2 g/N·m^3 and 2∿10 g/N·m^3 respectively. With the fluidized bed furnace, however, combustion residues are discharged entirely out of the furnace top. The combustion residues are precipitated by a cyclone dust collector, and the dust concentration at the furnace outlet is 30∿80 g/N·m^3.

Use of the cyclone dust collector is a suitable method of dust precipitation if the particles are of a size of 10μ or greater and in high concentration. With the combustion residue of a fluidized bed furnace, the cyclon dust collector has a precipitating efficiency of 80∿90%.

For the spray scrubber, a counter contact flow system of exhaust gas and water is employed. Where the dust concentration was 0.5∿2.0 g/N·m^3 and the water-to-exhaust gas ratio was 8 ℓ/m^3, the dust precipitating efficiency was 85%.

For the bentury scrubber, a parallel contact flow system of exhaust gas and water is employed. Where the dust concentration was 3∿10 g/N·m^3 and the water-to-exhaust gas ratio was 24 ℓ/m^3, the dust precipitating efficiency was 98∿99%.

The wet electric dust precipitator is designed for precipitation of dust particles of a size less than 1μ. With the dust concentration at 0.1 g/N·m^3, it provided a dust precipitating efficiency of 85∿89%.

Table A.3 shows the survey data of exhaust gas treatment equipment for multi-hearth furnace. The exhaust gas from the furnace outlet is scrubbed with water at the spray scrubber, then with sodium hypolorite and sodium hydroxide solutions at the chemical-absorption scrubber.

Offensive odor substances removed the water washing include the following:

Ammonia, acetaldehyde, trimethylamine and hydrogen sulfide.

Offensive odor substances removed by alkali scrubbing include the following:

Methyl mercaptan, hydrogen sulfide and methyl sulfide.

Offensive odor substances removed by acid scrubbing include the following:

Ammonia and trimethylamine.

Offensive odor substances removed by scrubbing with sodium hypochlorite solution include the following:

Methyl mercaptan, hydrogen sulfide and methyl sulfide.

Accordingly, as the exhaust gas at the furnace outlet is scrubbed with water, alkali, acid and sodium hypochlorite, the dust concentration, SOx and hydrogen chloride are reduced, while methyl mercaptan, hydrogen sulfide, methyl sulfide, trimethylamine and acetaldehyde are also removed, so that the odor concentrations emitted into atmospheric air range between 500 and 1500.

Table 1.1 Incinerator Installations in Japan

(March 1978)

Types of Furnace	Number	Design Capacity (ton·wet base/day)	Cake Moisture Content (%)
Multi-hearth furnace	68	5 ∿ 300	55 ∿ 80
Fluidized bed furnace	10	8 ∿ 50	65 ∿ 85
Rotary kiln	8	24 ∿ 36	60 ∿ 80
Stoker furnace	6	5 ∿ 43	47 ∿ 55
Others	5	-	-

Table 1.2 Results of Detailed Survey

Type of Furnace Name of Sewage Treatment Plant			Multi-heart Furnace			Fluidized Bed Furnace		Rotary Kiln	
	Items	Unit	S	A	N	Y	G	KN	KO
1	Design capacity	tons· wet base/day	300	40	50	40	50	24	36
2	Cake feed	kg/hr	12,550	1,530	1,843	1,425	1,725	1,055	1,438
3	Cake moisture content(wet base)	%	78.5	82.5	68.7	81.2	80.5	61.2	78.4
4	Dry cake lower calorific value	K Cal/kg. dry-base	2,690	2,500	2,430	3,560	2,000	1,980	3,280
5	Fuel consumption of heavy oil	ℓ/hr	498	113	49	95.5	175	*36.2	*71.1
6	Air supply	Nm^3/hr	32,100	5,230	4,420	2,330	3,360	3,280	6,060
7	Exhaust gas temperature at furnace outlet	°C	270	280	340	-	880	500	500
8	Exhaust gas temperature after heat exchange	°C	-	-	-	442	390	376	-
9	Wet volume of exhaust gas	Nm^3/hr	46,000	7,050	6,220	3,470	5,300	4,330	7,790
10	Dry volume of exhaust gas	Nm^3/hr	31,000	4,980	3,940	2,300	3,230	3,070	5,940
11	Ash discharge	kg/hr	1,100	122	300	75	168	161	96
12	Combustion temperature	°C	850	750	826	874	900	**800	**900
13	Overall air supply rate	-	2.8	2.9	2.2	1.2	1.4	2.4	3.2
14	Furnace wall and other unidentifiable heat loss/Input heat	%	16	18	22	29	9	24	15

* Assumed fuel consumption converted from city gas based on lower calorific value.

** Assumed combustion temperature estimated from the formula of heat conduction upon measurement of the surface temperature.

Table 2.1 Heat Balance of Multi-hearth Furnace
(300 tons·wet base/day)

20°C base

Input Heat		Output Heat	
K cal/hr	%	K cal/hr	%
(1) Combustion heat of dewatered cake 7.252×10^6	67.4	(1) Exhaust gas potential heat 3.844×10^6	35.7
(2) Combustion heat of A-type havy oil 3.504×10^6	32.6	(2) Cake moisture evaporation latent heat 5.773×10^6	53.7
		(3) Ash potential heat 0.009×10^6	0.1
		(4) Radiant conduction and other heat loss 1.130×10^6	10.5
10.756×10^6	100.0		100.0

Fig. 2.1 Flow Diagram of Multi-hearth Furnace
(300 tons·wet-base/day)

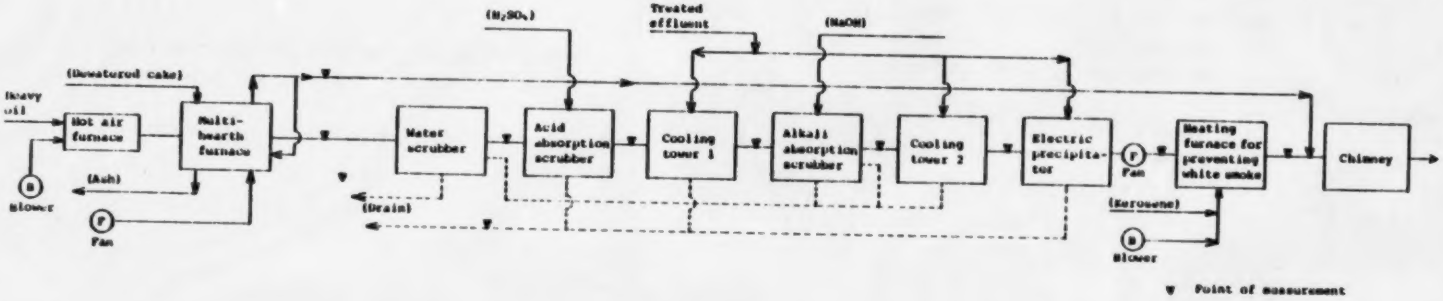

(H₂SO₄)
Treated effluent
(NaOH)
(Dewatered cake)
Heavy oil
Hot air furnace
Multi-hearth furnace
Water scrubber
Acid absorption scrubber
Cooling tower 1
Alkali absorption scrubber
Cooling tower 2
Electric precipitator
Fan
Heating furnace for preventing white smoke
Chimney
Blower
(Ash)
Fan
(Drain)
(Kerosene)
Blower
▼ Point of measurement

Fig. 2.2 Material Balance of Multi-hearth Furnace
(300 tons·wet base/day)

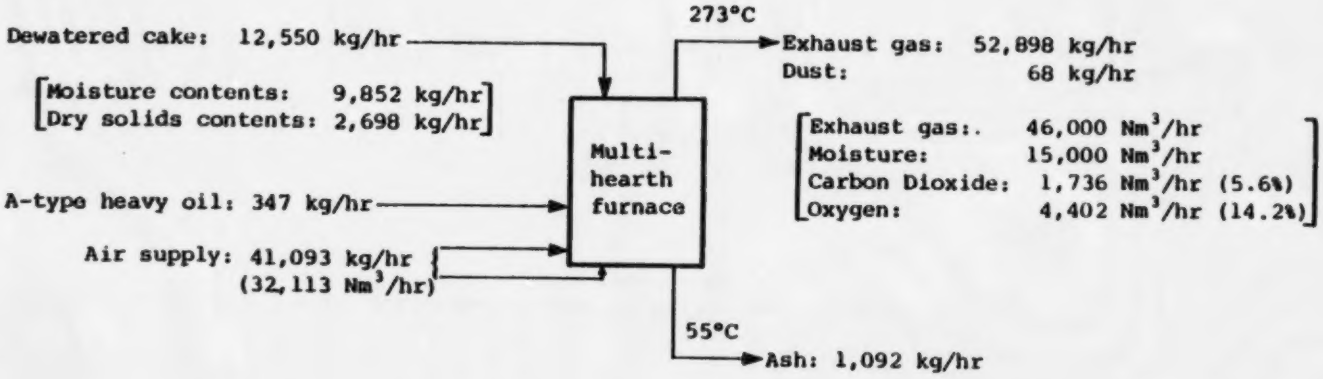

Dewatered cake: 12,550 kg/hr
Moisture contents: 9,852 kg/hr
Dry solids contents: 2,698 kg/hr
A-type heavy oil: 347 kg/hr
Air supply: 41,093 kg/hr
(32,113 Nm³/hr)
Multi-hearth furnace
273°C
Exhaust gas: 52,898 kg/hr
Dust: 68 kg/hr
Exhaust gas:. 46,000 Nm³/hr
Moisture: 15,000 Nm³/hr
Carbon Dioxide: 1,736 Nm³/hr (5.6%)
Oxygen: 4,402 Nm³/hr (14.2%)
55°C
Ash: 1,092 kg/hr

Table 3.1 Heat Balance of Fluidized Bed Furnace
(50 tons·wet base/day)

(Furnace and to Heat Exchanger Outlets)

Input Heat		Output Heat	
K cal/hr	%	K cal/hr	%
(1) Combustion heat of dewatered cake 6.72×10^5	31.2	(1) Exhaust gas potential heat 5.91×10^5	27.4
(2) Combustion heat of A-type heavy oil 14.84×10^5	68.8	(2) Cake moisture evaporation latent heat 8.13×10^5	37.7
		(3) Ash and dust potential heat 0.14×10^5	0.7
		(4) Discharge heat for white smoke prevention 3.05×10^5	14.1
		(5) Radiant conduction and other heat loss 4.33×10^5	20.1
21.56×10^5	100.0	21.56×10^5	100.0

Fig. 3.1 Flow Diagram of Fluidized Bed Furnace
(50 tons·wet base/day)

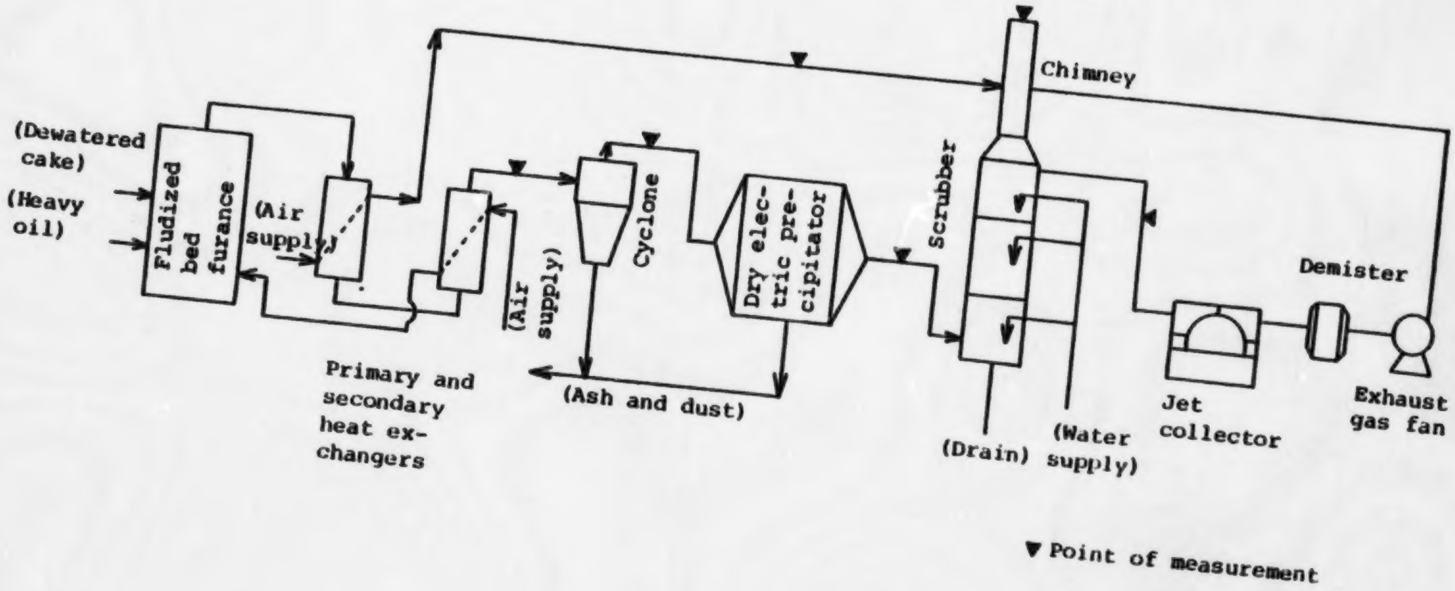

(Dewatered cake)
(Heavy oil)
Fluidized bed furance
(Air supply)
(Air supply)
Primary and secondary heat ex-changers
Cyclone
Dry elec-tric pre-cipitator
(Ash and dust)
Scrubber
Chimney
(Drain)
(Water supply)
Jet collector
Demister
Exhaust gas fan
▼ Point of measurement

Fig. 3.2 Material Balance of Fluidized Bed Furnace
(50 tons·wet base/day)

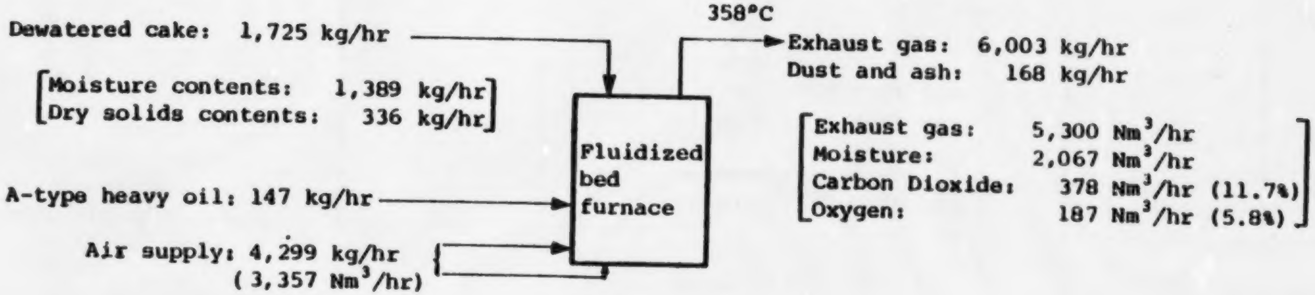
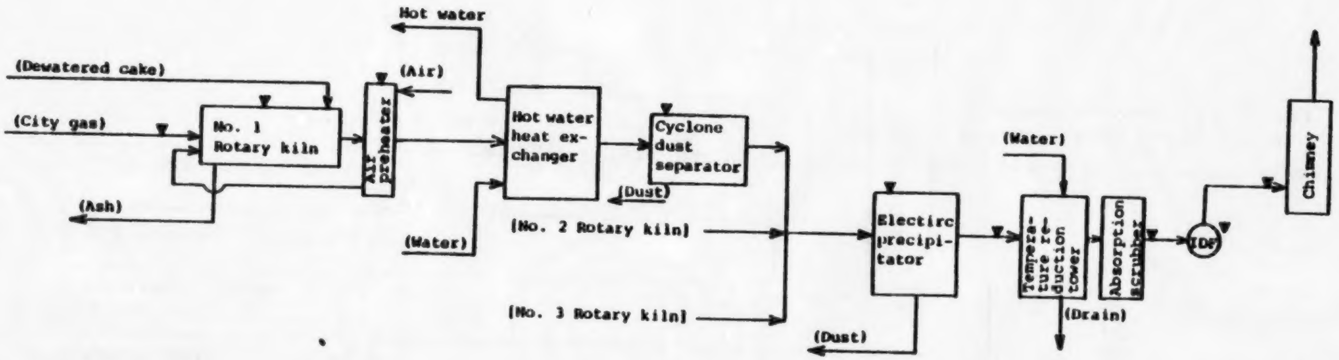

Dewatered cake: 1,725 kg/hr
Moisture contents: 1,389 kg/hr
Dry solids contents: 336 kg/hr
A-type heavy oil: 147 kg/hr
Air supply: 4,299 kg/hr
(3,357 Nm³/hr)
Fluidized bed furnace
358°C
Exhaust gas: 6,003 kg/hr
Dust and ash: 168 kg/hr
Exhaust gas: 5,300 Nm³/hr
Moisture: 2,067 Nm³/hr
Carbon Dioxide: 378 Nm³/hr (11.7%)
Oxygen: 187 Nm³/hr (5.8%)

Table 4.1 Heat Balance of Rotary Kiln (24 tons·wet base/day)

Input heat	$\times 10^3$ K cal/hr	%	Output heat	$\times 10^3$ K cal/hr	%
(1) Combustion heat of dewatered cake	810	71.2	(1) Potential heat of theoretical wet base exhaust gas	196	17.2
(2) Combustion heat of city gas	327	28.8	(2) Potential heat of excessive air	117	10.3
(3) Heat air acquired at air preheater	(72)	(6.3)	(3) Potential heat of penetrating air	103	9.1
			(4) Potential heat of evaporation of cake moisture	101	8.9
			(5) Latent heat of evaporation of cake moisture	386	33.9
			(6) Potential heat of ash	15	1.3
			(7) Radiant heat of kiln	167	14.7
			(8) Unidentifiable heat	52	4.6
			(9) Heat exhaust gas emitted at air preheater	(72)	(6.3)
Total	1,137	100.0	Total	1,137	100.0

(Note) Parenthesized data represent heat circulation.

Fig. 4.1 Flow Diagram of Rotary Kiln
(24 tons·wet base/day)

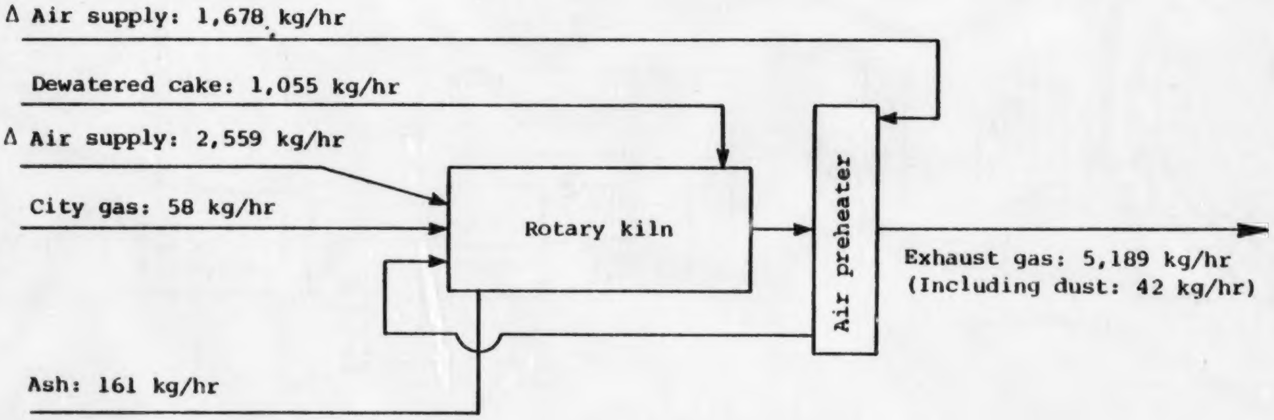


Hot water
(Dewatered cake)
(Air)
(City gas)
No. 1 Rotary kiln
Air preheater
Hot water heat exchanger
Cyclone dust separator
(Dust)
(Ash)
(Water)
(No. 2 Rotary kiln)
(No. 3 Rotary kiln)
Electirc precipitator
(Dust)
(Water)
Temperature reduction tower
(Drain)
Absorption scrubber
IDF
Chimney

Note: Nos. 2 ∿ 3 Rotary kilns are of the same type as No. 1 Rotary kiln under measurement.

▼ Point of measurement

Fig. 4.2 Material Balance of Rotary Kiln
(24 tons·wet base/day)


Δ Air supply: 1,678 kg/hr
Dewatered cake: 1,055 kg/hr
Δ Air supply: 2,559 kg/hr
City gas: 58 kg/hr
Rotary kiln
Air preheater
Exhaust gas: 5,189 kg/hr
(Including dust: 42 kg/hr)
Ash: 161 kg/hr

(Note) Λ: Estimated calculation

Table 6.1 Primary Units of Utilities for Dewatering and Incineration During Steady Burning* (per ton of dry solid)

Items			Fuel		Electric power			Chemicals				Water			Miscellaneous
			A-type heavy oil	Kerosene**	For de-watering	For burning	Total	Lime	Ferric chloride	Polymer	Caustic soda	For de-watering	For incineration	Total	Silica sand
Incinerators	Dewatering Devices	Moisture contents	ℓ/ton· dry solid	ℓ/ton· dry solid	KWH/ton· dry solid	KWH/ton· dry solid	KWH/ton· dry solid	kg/ton· dry solid	kg/ton· dry solid	kg/ton· dry solid	kg/ton· dry solid	m^3/ton· dry solid	m^3/ton· dry solid	m^3/ton· dry solid	kg/ton· dry solid
Multi-hearth furnace	Vacuum filter	77%	210	340	80	310∿410	390∿490	350	180	-	20	40	140∿150	180∿190	-
	Filter press	65%	0	280∿290	60∿80	240∿360	300∿440	350	180	-	10	10	90	100	-
	Centrifuge	80%	130	290	140	250∿380	390∿520	-	-	10	50	5∿10	120∿130	130∿140	-
Fluidized bed furnace	Vacuum filter	77%	480	-	80	430∿600	510∿680	350	180	-	30	40	160∿170	200∿210	30
	Filter press	65%	120	-	60∿80	280∿380	330∿460	350	180	-	20	10	90∿100	110∿120	30
	Centrifuge	80%	340∿350	-	140	340∿540	470∿680	-	-	10	50∿60	5∿10	140∿150	140∿150	20
Rotary kiln	Vacuum filter	77%	340	310	80	340∿460	420∿540	350	180	-	30	40	180∿190	220∿230	-
	Filter press	65%	80	240	60∿80	260∿380	320∿460	350	180	-	20	10	140∿150	160	-
	Centrifuge	80%	250	290	140	340∿460	480∿600	-	-	10	50	5∿10	160∿180	170∿180	-

* Sewage flow is within the range of 50,000∿200,000 m^3/day.
** Kerosene is for direct burning and deodorizing.

Table 6.2 Utility Cost Required for Steady Burning
(Japanese Yen per ton of dry solid)

(February 1979)

(Yen)

	Items	Fuel	Electric power	Chemical	Water	Miscellaneous	Total
Incinerators	Dewatering Devices	A-type heavy oil	-	Caustic soda	Industrial water	Silica sand	
Multi-hearth furnace	Vacuum filter	7,525	3,600	1,120	2,175	-	14,400
	Filter press	0	3,000	560	1,350	-	4,900
	Centrifuge	4,375	3,150	2,800	1,875	-	12,200
Fluidized bed furnace	Vacuum filter	16,975	5,150	1,680	2,475	750	27,000
	Filter press	4,200	3,250	1,120	1,425	750	10,700
	Centrifuge	12,075	4,400	3,080	2,175	500	22,200
Rotary kiln	Vacuum filter	12,075	4,000	1,680	2,775	-	20,500
	Filter press	2,975	3,150	1,120	2,175	-	9,400
	Centrifuge	8,925	4,000	2,800	2,550	-	18,300

Table 6.3 Proportions for Dewatering Equipment and Incinerators in Equipment Costs

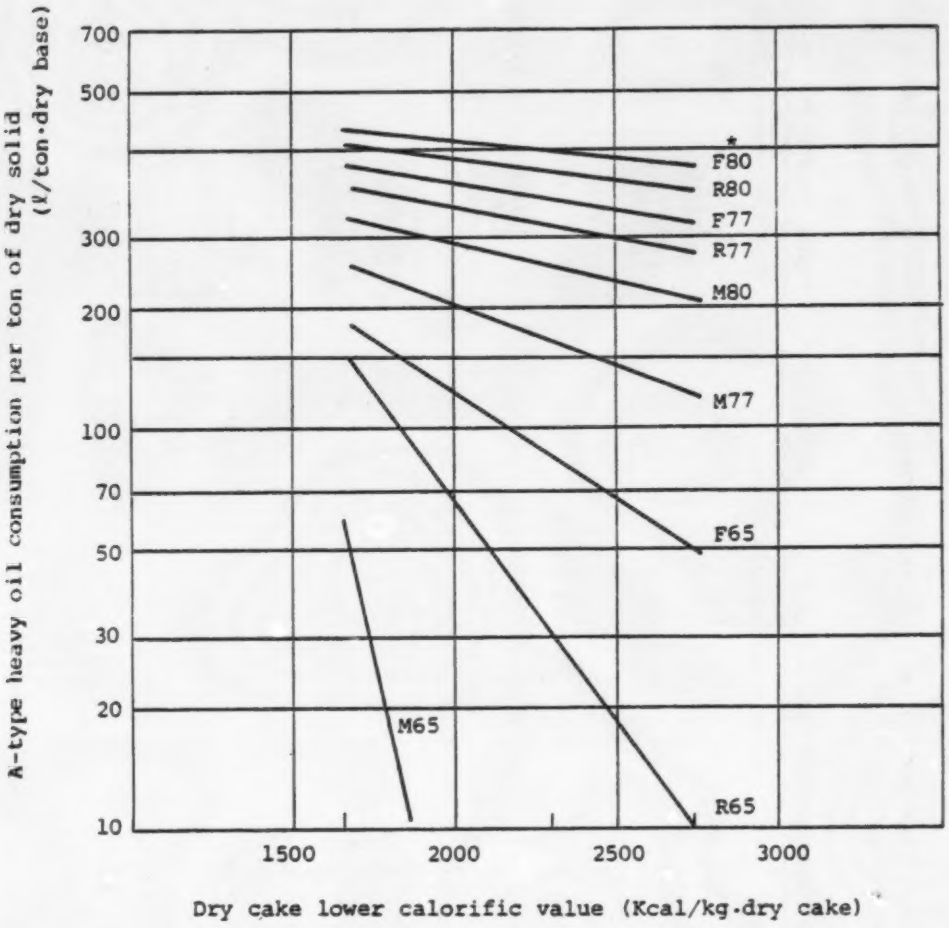
Incinerators	Sewage flow Equipments	50,000 m^3/day	125,000 m^3/day	200,000 m^3/day
Multi-hearth furnace	Vacuum filter	27%	32%	32%
	Incinerator	73%	68%	68%
	Total	100%	100%	100%
	Filter press	54%	62%	67%
	Incinerator	46%	38%	33%
	Total	100%	100%	100%
	Centrifuge	30%	35%	34%
	Incinerator	70%	65%	66%
	Total	100%	100%	100%
Fluidized bed furnace	Vacuum filter	26%	34%	33%
	Incinerator	74%	66%	67%
	Total	100%	100%	100%
	Filter press	52%	61%	69%
	Incinerator	48%	39%	31%
	Total	100%	100%	100%
	Centrifuge	29%	35%	35%
	Incinerator	71%	65%	65%
	Total	100%	100%	100%
Rotary kiln	Vacuum filter	30%	37%	36%
	Incinerator	70%	63%	64%
	Total	100%	100%	100%
	Filter press	57%	65%	71%
	Incinerator	43%	35%	29%
	Total	100%	100%	100%
	Centrifuge	33%	39%	38%
	Incinerator	67%	61%	62%
	Total	100%	100%	100%

Table 6.4 Approximate Occupancy Areas of Dewatering Equipment and Incinerators

(Unit: m^2)

Incinerators	Dewatering Devices and Incinerators / Sewage flow	Vacuum filter	Incinerator	Total	Filter press	Incinerator	Total	Centrifuge	Incinerator	Total
Multi-hearth furnace	50,000 m^3	650	3,190	3,840	920	2,880	3,800	580	2,880	3,460
	125,000 m^3	1,520	4,600	6,120	1,540	4,600	6,140	1,010	5,040	6,050
	200,000 m^3	2,170	6,900	9,070	2,630	4,600	7,230	1,340	6,900	8,240
Fluidized bed furnace	50,000 m^3	650	1,920	2,570	920	1,800	2,720	580	1,800	2,380
	125,000 m^3	1,520	2,630	4,150	1,540	2,630	4,170	1,010	3,070	4,080
	200,000 m^3	2,170	3,940	6,110	2,630	2,630	5,260	1,340	3,940	5,280
Rotary kiln	50,000 m^3	650	3,190	3,840	920	2,880	3,800	580	2,880	3,460
	125,000 m^3	1,520	4,600	6,120	1,540	4,600	6,140	1,010	5,040	6,050
	200,000 m^3	2,170	6,900	9,070	2,630	4,600	7,230	1,340	6,900	8,240

Fig. 6.1 A-type Heavy Oil Consumption versus Dewatered Cake Calorific Value and Moisture Content



*
Note 1 M: Multi-hearth furnace
F: Fluidized bed furnace
R: Rotary kiln

Note 2 Numerical figures show the moisture contents (%) of feed cake.

Fig. 6.2 Utility Cost Required for Dewatering and Incineration during Steady Burning (Japanese Yen per ton of dry solid)

(February 1979)

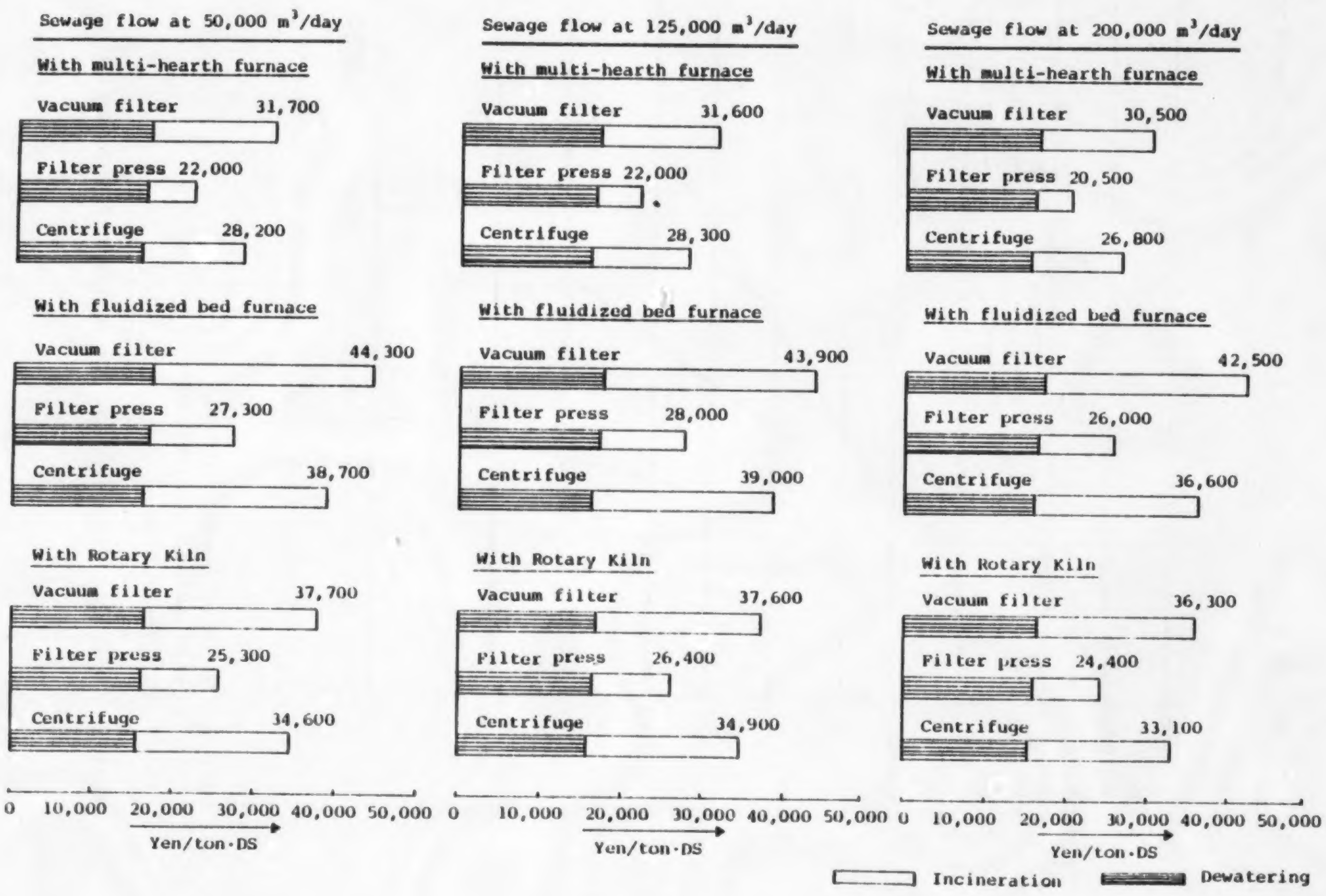


Fig. 6.3 Approximate Cost Indexes of the Dewatering and Incineration Facilities

(February 1979)

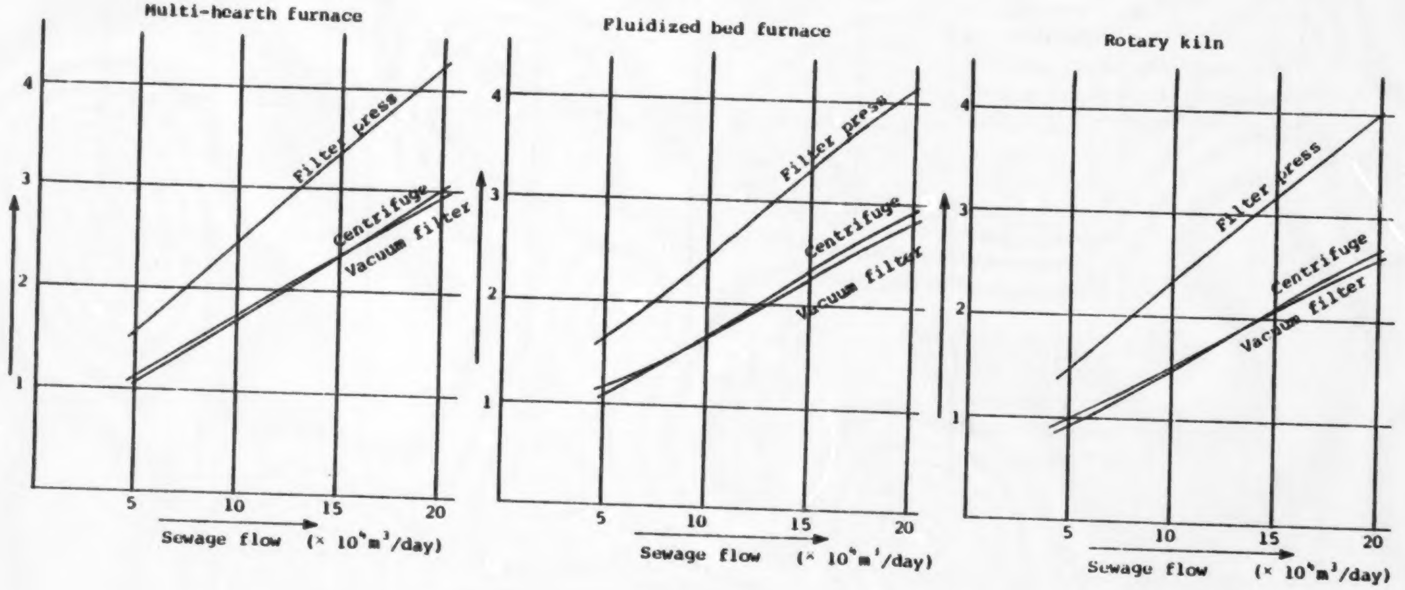


Fig. 6.4 Approximate Occupancy Areas for Dewatering and Incineration Facilities

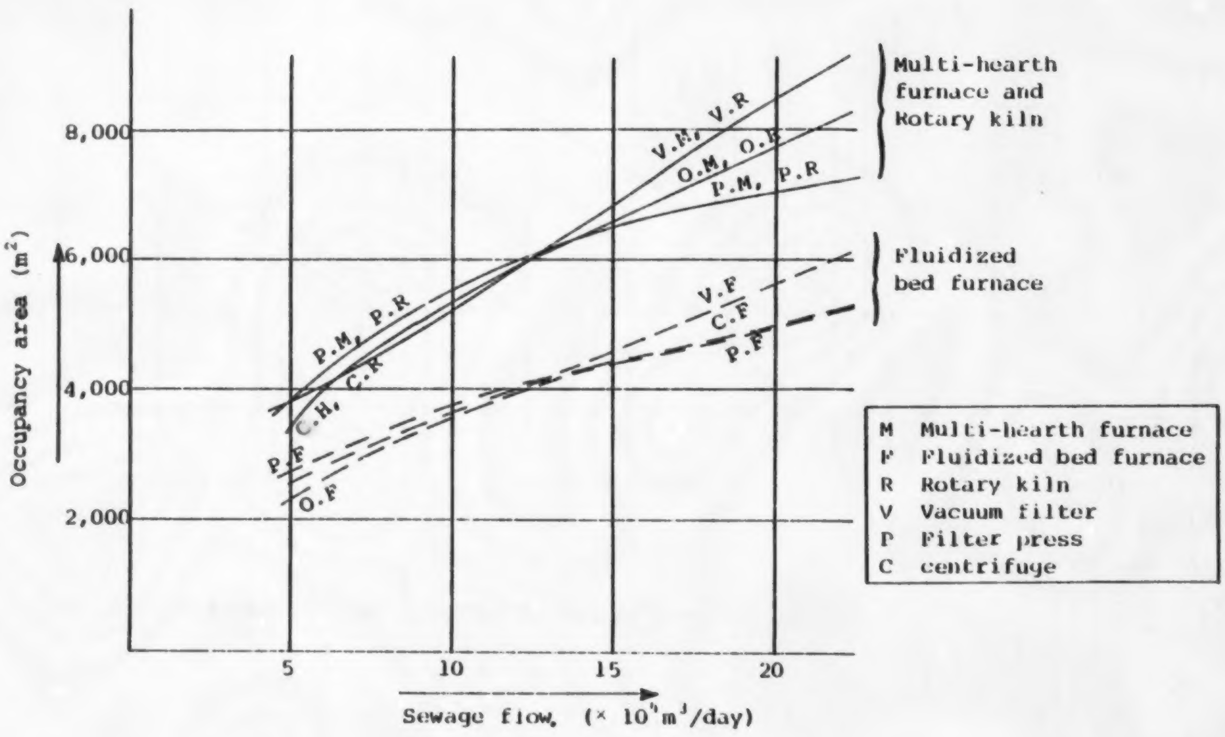


Table A.1 Characteristics of Raw Sludge Cake

Sewage Treatment Plants	Chemicals	Dewatering Devices	No. of Analysis	VSS/DS (%)	Calorific Values	C	H	N	S	Sulfur Combustible
FG	Ferric Chloride Lime	Vacuum filter	3	53.0	3,163	26.4	5.4	4.3	0.62	-
GA	Ferric Chloride Lime	"	1	46.9	2,680	27.1	4.0	3.7	0.90	0.1
MS	"	"	2	49.5	2,980	27.6	4.3	3.7	0.91	0.1
MH	"	"	2	46.7	2,790	26.3	3.9	3.1	1.0	-
BT	"	"	2	47.4	2,860	27.5	4.1	3.1	1.3	-
MN	"	Filter Press	3	40.1	2,340	23.4	4.1	3.0	1.7	0.1
HS	"	"	1	38.2	1,960	16.7	3.6	2.1	0.73	-
EI	"	"	1	39.2	2,190	23.2	3.9	3.3	0.93	0.0
SF	"	"	3	38.1	2,110	21.5	3.3	3.3	2.2	0.7
MI	"	"	1	38.3	2,310	21.8	4.2	2.8	1.2	0.0
SYc	Polymer	Centrifuge	2	66.4	3,890	36.5	4.8	5.7	1.0	-
SYk	"	"	4	49.9	2,980	27.6	4.2	2.6	1.6	1.5
BM	"	"	1	68.1	4,030	37.5	5.5	3.9	1.0	-
DM	"	"	1	84.7	4,690	41.9	6.8	5.7	1.1	-
EI	"	Belt Press	1	61.5	3,420	32.3	4.9	3.9	1.0	0.9
SB	Ferric Chloride Per-oxide	Vacuum filter	1	73.2	4,440	39.5	5.6	5.7	2.8	1.4
BK	Heat treatment	Filter press	11	78.1	4,430	40.5	5.7	2.9	0.70	-
RK	"	"	4	63.7	4,030	37.8	5.3	2.8	1.1	0.2
SF	-	-	1	53.7	2,550	23.1	3.8	4.0	2.7	1.8
KM	-	-	1	51.2	2,950	27.4	3.5	3.2	-	0.8
MS	-	-	1	44.6	2,750	24.5	3.3	2.7	-	1.0
US	-	-	1	42.7	2,560	24.0	3.0	2.8	-	0.8
DO	-	-	1	51.7	2,960	28.8	3.1	3.0	-	0.6
AO	-	-	1	47.0	2,630	24.8	2.9	3.3	-	0.6

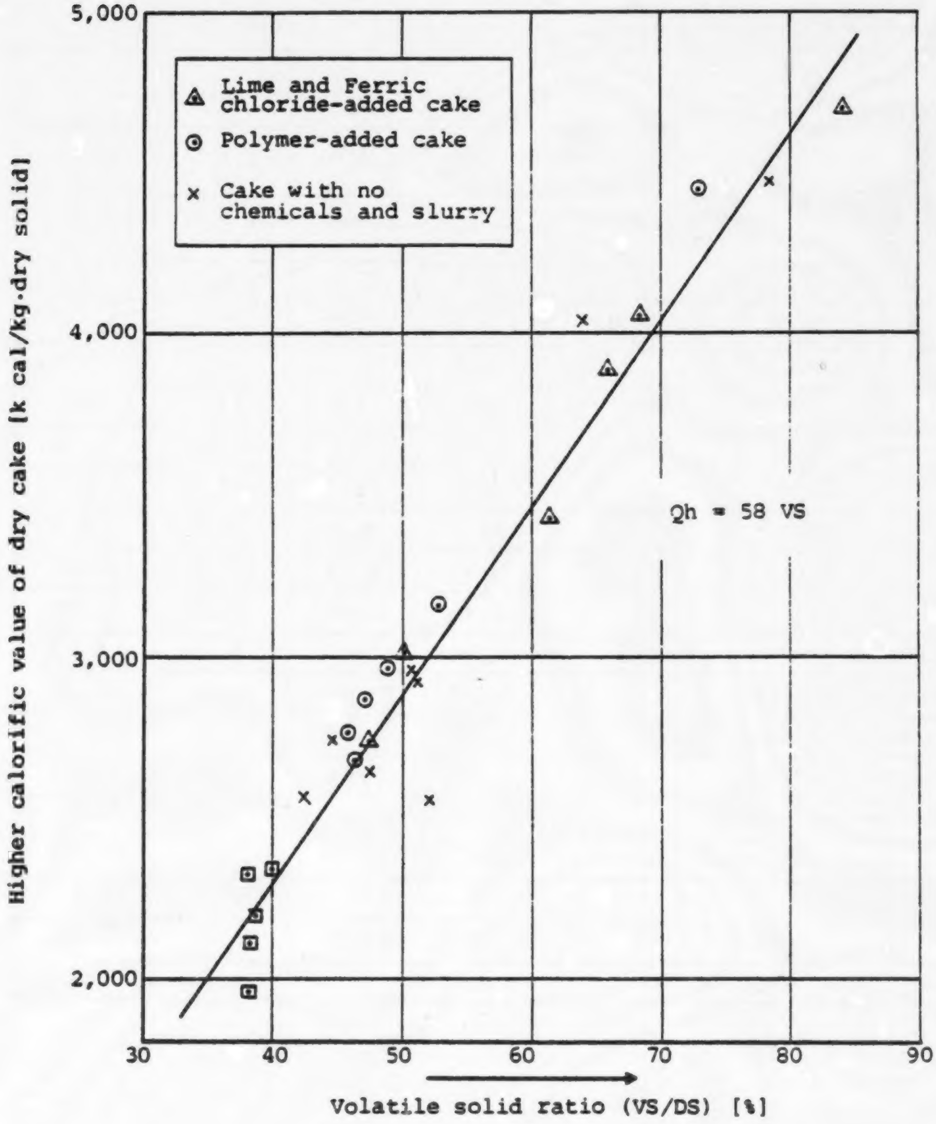
Table A.2 Theoretical Air and Combustion Gas Volume per Unit Dewatered Cake Volatile Solid (kg)

Kinds of Dewatered Cake	Items / Gas Composition / Unit	Theoretical Combustion Gas Volume			Theoretical Air Volume (Lo)
		Wet Base (Go)	Dry Base (Go')		
		$Nm^3/kg \cdot VS$	$Nm^3/kg \cdot VS$	% or ppm	$Nm^3/kg \cdot VS$
Lime added cake	N_2 (Air)	4.21	4.21	-	5.93
	CO_2	1.01	1.01	19.1 %	
	H_2O	0.905	0	0	
	SO_2	0.0015	0.0015	280 ppm	
	NO	0.0022	0.0022	420 ppm	
	N_2 (Volatile solid)	0.0543	0.0543	-	
	Total	6.18	5.27	100 %	
Polymer added cake and cake with no chemicals	N_2 (Air)	4.19	4.19	-	5.90
	CO_2	0.99	0.99	18.8 %	
	H_2O	0.90	0	0	
	SO_2	0.014	0.014	2,600 ppm	
	NO	0.0022	0.0022	410 ppm	
	N_2 (Volatile solid)	0.053	0.053	-	
	Total	6.15	5.25	100 %	

Table A.3 Performances of Exhaust Gas Treatment Equipments in a Multi-hearth Incinerator

Items of Measurement / Point of Measurement	Unit	Furnace Outlet	Spray Scrubber Outlet	Chemical-Absorption Scrubber Outlet	Chimney Outlet
Exhaust gas temperature	°C	232	28	30	68
Exhaust gas volume (wet)	Nm^3/H	11,350	9,540	9,960	12,495
Exhaust gas volume (dry)	"	8,680	9,170	9,535	11,910
Moisture content	%	23.5	3.9	4.3	4.7
Dust	g/Nm^3	0.545	0.138	0.070	0.034
CO_2	%	6.7	6.2	6.0	4.2
O_2	"	12.1	13.0	13.5	15.8
CO	"	0.0	0.0	0.0	0.0
SOx	ppm	55.8	11.1	9.8	4.3
NOx	"	99.4	85.1	81.0	71.8
$HC\ell$	"	120.07	24.84	17.65	10.34
$C\ell_2$	"	1.46	1.46	6.11	2.24
Ammonia	"	131.54	10.80	8.15	3.83
Trimethylamine	"	0.297	0.123	0.067	0.028
Hydrogen sulfide	"	1.300	0.436	0.396	0.235
Methyl sulfide	"	0.143	0.052	0.034	0.017
Methyl mercaptan	"	0.473	0.242	0.182	0.114
Dimethyl disulfide	"	0.0005	0.0004	0.0002	0.0001
Acetaldehyde	"	<1.0	<1.0	<1.0	<1.0
Styrene monomer	"	<0.001	<0.001	<0.001	<0.001
Odor Concentration	"	6,900		1,400	

Fig. A.1 Volatile Solid Ratio VS. Upper Calorific Values in Various Dewatered Cake and Slurry



Seventh US/JAPAN Conference
on
Sewage Treatment Technology

PUBLICLY OWNED TREATMENT WORKS PRETREATMENT CONTROL PRACTICE IN TOKYO

May 20, 1980
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PUBLICLY OWNED TREATMENT WORKS
PRETREATMENT CONTROL PRACTICE IN TOKYO

1. INTRODUCTION 170
    - 1.1 Scale of Sewer System in the Inner City of Tokyo 170
    - 1.2 Example of Interference with the Wastewater Treatment Plant caused by Industrial Wastewater 170
        - 1.2.1 Copper 170
        - 1.2.2 Cyanides 171
        - 1.2.3 Oils 171
        - 1.2.4 Synthetic detergent 171

2. STATUS OF INDUSTRY EFFLUENT LIMITATIONS 173
    - 2.1 Method of Control 173
        - 2.1.1 Regulation by penalties 173
        - 2.1.2 Regulation by an obligation for installation of pretreatment facility 173
    - 2.2 Specified Industry Subject to Guidance and Average Wastewater Quality Classified by Industry-by-industry 175
    - 2.3 Situation of Pretreatment Facility Installed 175
    - 2.4 Surveillance and Guidance Program for Industrial Wastewater 176
        - 2.4.1 Structure of execution 176
        - 2.4.2 Status of surveillance 177
        - 2.4.3 Steps for regulation of industrial wastewater 177
        - 2.4.4 System for nomination of person responsible for wastewater quality control .. 178

3. FINDING AND GUIDANCE TO THE INDUSTRY NOT INSTALLED THE PRETREATMENT FACILITY 180

4. LOAN SYSTEM 180

5. MAINTENANCE OF THE PRETREATMENT FACILITY 182

6. RECENT TREND OF THE PRETREATMENT INSTALLATION 183
    - 6.1 General Trend 183
        - 6.1.1 Treatment process 183
        - 6.1.2 Utilization of recoveries 183
        - 6.1.3 Treatment of copper and zinc 183
        - 6.1.4 Prepackaged unit treatment 183
        - 6.1.5 Dewatering of sludge 183
    - 6.2 Features of Treatment Classified by Industry-by-industry 185
        - 6.2.1 Foods processing industry 185
        - 6.2.2 Textile industry 185
        - 6.2.3 Metal products manufacturing 185
        - 6.2.4 Machinery manufacturing (1) – (with metal-plating process) 186
        - 6.2.5 Machinery manufacturing (2) – (without metal-plating process) 186
        - 6.2.6 Regular metal-plating industry 186
        - 6.2.7 Plate making and printing 186
        - 6.2.8 Chemical industry 186
        - 6.2.9 Ceramics industry 186
        - 6.2.10 Others 186

6.3 Example of a Joint Treatment of Wastewater Discharged from Collective Minor-sized Factories 187
  6.3.1 Structure of facility 188
  6.3.2 Joint treatment facility of wastewater 188
  6.3.3 Recycling for reuse of industrial wastewater 190
6.4 Example of a Joint Treatment of Highly-concentrate Cyanide Wastewater 190
  6.4.1 Objective items for treatment 190
  6.4.2 Treatment method 190
  6.4.3 Status of treatment 193

[Reference Materials]

1. Letter Requesting for Investigation on Wastewater 194
2. Inquiry Sheet for Industrial Wastewater 194
3. Ledger of Factory Subject to Guidance 194

1. INTRODUCTION

1.1 Scale of Sewer System in the Inner City of Tokyo

A sewer system is presently serving approximately 70% of the inner city of Tokyo that comprises of 23 Wards, but when viewed regionally, the spread in the eastern area of Arakawa River is somehow backward.

Overall scheme is planned to divide the inner city of Tokyo into 10 regional wastewater treatment districts depending on its topographical features in order to equalize the quantitative loads mutually. When the scheme is complete, the district's common sewer system can provide for about 10.36 million people and will have a capacity of treating 9.79 million cubic meters per day. The eight wastewater treatment plants are presently operating and their treatment capabilities are 5,453,000 m^3/day now.

A densely industrial area, when viewed from the geographical distribution of factories, is located in both sides of Shingashi River, nearby area of Mikawashima Wastewater Treatment Plant, Suna-machi, Yotsugi, both sides of Meguro River and the area along Tama River. In these areas, the industries consist mostly of minor-sized factories such as textiles, metal-plating, plate making for printing, metal products manufacturing, etc. Food processing, iron and steel manufacturing, and chemicals manufacturing in the area are comparatively large-sized industries, but, in terms of number of factory, these comprise of less than 10% of the categorized industry.

The categorized or specific industry represents factory and establishment that are within the bounds of possibility of discharging wastewater limited by the Sewerage Law or Sewerage Ordinance of Tokyo Metropolitan Government into a Publicly Owned Treatment Works (POTW) system. The industry of this category is discharging, on average, about 390,000 m^3/day of total volume received by all regional wastewater treatment plants in Tokyo.

1.2 Example of Interference with the Wastewater Treatment Plant caused by Industrial Wastewater

Some of interferences with the sewage treatment plant caused by industrial wastewater in the past are presented as follows.

1.2.1 Copper (experienced at Mikawashima Plant in 1973)

a. Influent and influence on the plant

Sudden deterioration of the secondary effluent quality was involved, resulting in lowering a 40 cm of transparency down to 10 cm. An exceptional inflow of copper was found when the survey of the effluent quality variation with time was conducted for cause-finding. The inflow of wastewater containing concentrations of higher than 1 mg/ℓ extended over four hours at intervals of 30 to 60 minutes intermittently. The maximum concentrations of copper in stream were 2.1 mg/ℓ. It was assumed that the degradation of the effluent quality was caused by lowered effect of the activated sludge process due to the presence of copper.

Investigation on the trunk sewer was made to search after a point of discharger. The search was reached upto a lateral sewer, however, an industry discharged such wastewater was not identified.

b. Measures

The activation of the activated sludge process was restored by seeding continuously normal active sludge introduced from another source into an aeration tank.

Henceforth, the regulation for discharge of copper and zinc was also imposed on a factory that discharges wastewater of less than 50 m^3/day, since the amendment to the Regulations for Sewerage Ordinance Enforcement of Tokyo Metropolitan Government was enacted in February, 1976.

1.2.2 Cyanides (experienced at Mikawashima Plant in 1974)

a. Influent and influence on the plant

In the early morning, a colloidal precipitation in the effluent was identified. The effluent quality was examined and 0.5 mg/ℓ of cyanides was detected in the effluent. Considering of timing of discovery, it was assumed that cyanides in higher concentrations inflowed during night time.

The colloidal precipitation was caused by the residues of fine particles, resulting from a partial destruction of activated sludge.

b. Measures

Investigation on the trunk sewer was conducted immediately after such finding, however, a source of discharge was not identified since a sudden inflow of cyanides passed through in a short period.

Alteration to the operation mode was not made, but the effluent quality returned to a normal on the following day.

1.2.3 Oils (seen at Shibaura Plant in 1973)

a. Influent and influence on the plant

In the treatment plant that received wastewater mixed with wastes from slaughterhouse, a lot of forth of dark-yellowish scums were formed in the aeration tank and final sedimentation tank. The scums were formed of froth that was derived from surfactant, oil droplet and activated sludge floc. Oil contents in the scums were about 6% as dry weight.

b. Measures

Oil fence was installed at the inlet of final sedimentation tank. This prevented the scums from flowing over the weir and residual scums were collected for disposal.

The slaughterhouse modified the discharge line of wastewater and a plain oil separator was changed to the flotation tank employing dispersed air. In addition, the plant for activated sludge process was improved.

1.2.4 Synthetic detergent (seen at Mikawashima and Ochiai Plants in 1974)

a. Influent and influence on the plant

Inflow of ABS, that is a chief ingredient of synthetic detergent for domestic use, generated froth in the aeration tank on such occasion that a temperature was low, decomposition ability of activated sludge was degraded, or MLSS concentrations were lowered. The froth was also found at an outfall. This was due to the residue of ABS in the effluent. The froth was scattered around by windage, which sometimes interfered with inhabitants living in the immediate neighborhood. Residual concentrations of ABS (sodium alkyl benzene sulfonate) in the effluent from the wastewater treatment plant are shown in Table 1.1.

Table 1.1 Concentration of sodium alkyl benzene sulfonate (ABS) (Average of 1974)

Sewage Treatment Plant	Influent (mg/ℓ)	Treated water (mg/ℓ)
Mikawashima (Asakusa District)	4.4	0.9
Mikawashima (Ogu District)	5.1	0.9
Mikawashima (Aizome District)	4.8	0.6
Ochiai	5.6	0.8

b. Measures

The steps to increase MLSS concentrations and decrease amount of supplying air were employed in the aeration tank as the measures. As another measures for prevention of froth growth, antifoaming agent such as silicon was fed into a defoaming tank and the polystyrene form plates were installed to allow absorption of the froth.

The products of synthetic detergent was turned to a soft-type in 1974. This allowed a biodegrability of the synthetic detergent of more than 85%, thus resulting in a remarkable reduction of froth forming.

2. STATUS OF INDUSTRY EFFLUENT LIMITATIONS

The effluent limitations set forth in the Sewerage Law and Sewerage Ordinance of Tokyo Metropolitan Government are applicable to those industries or factories that are located in the regional wastewater treatment districts and discharge wastewater into the public sewer.

2.1 Method of Control

The control or regulating can be broadly classified into two categories.

2.1.1 Regulation by penalties

The specified industry that is categorized under the statutory regulation is prohibited to discharge wastewater quality in excess of the effluent limitations described in Table 2.1. Any industry that violates any ordinance or regulation shall be prosecuted and, in some cases, punished directly. In the discretion of the General Manager of Publicly Owned Treatment Works system, an order for facility improvement or order for suspension of discharging wastewater can be issued without dispute to those in violation or those who possibly violate the regulation.

2.1.2 Regulation by an obligation for installation of pretreatment facility

The regulation imposes an obligation to install the pretreatment facility to the dischargers of wastewater in excess of the effluent limitations stated in Table 2.1. The order for facility improvement or suspension of discharge can be published to those in violation of the regulation.

Table 2.1 Pretreatment standards for industrial users of POTW

Pollutants or Items			Categories: Dischargers to Publicly Owned Treatment Works system where terminal treatment plant is installed.				
			Industry installed specified facility			Industry not installed specified facility	
			50 m^3/d or over	50 m^3/d or less		50 m^3/d or over	50 m^3/d or less
Harmful substances	Cadmium		0.1	0.1		0.1	0.1
	Cyanide		1	1		1	1
	Organic phosphorus		1	1		1	1
	Lead		1	1		1	1
	Chromium (VI)		0.5	0.5		0.5	0.5
	Arsenic		0.5	0.5		0.5	0.5
	Total mercury (*)		0.005	0.005		0.005	0.005
	Alkyl mercury		not detectable	not detectable		not detectable	not detectable
	P C B		0.003	0.003		0.003	0.003
Living environment item	Total chromium		2	2	2	2	2
	Copper		3	3	3	3	3
	Zinc		5	5	5	5	5
	Phenol		5	5	-	5	-
	Iron		10	10	-	10	-
	Manganese		10	10	-	10	-
	Fluorine		15	15	-	15	-
	B O D		600 (300)	-		600 (300)	-
	S S		600 (300)	-		600 (300)	-
	Normal-hexane	mineral oil	5	-		5	-
		animal/vegetable oil	30	-		30	-
	p H		5 – 9 (5.7 – 8.7)	5 – 9 (5.7 – 8.7)		5 – 9 (5.7 – 8.7)	5 – 9 (5.7 – 8.7)
	Temperature		45°C (40°C)	45°C (40°C)		45°C (40°C)	45°C (40°C)
	Iodine demand		220	220		220	220

(Note)
1. Unit, except for pH, is expressed in mg/ℓ.
2. Value in bracket ( ) related to BOD, SS, pH and Temperature is applicable to manufacturers and gas suppliers.
3. Pretreatment standards expressed in solid lined square ▨ are directly related to the penalties by regulations. Among these limitations, the standards of copper, zinc, phenol, iron, maganese, fluorine related to the industries installed specific facilities have been applicable to the factories constructed after April 2, 1972.
4. Indication in plain square ☐ relates to the obligation of pretreatment facility installation.

(*) Total mercury represents mercury, alkylmercury and other mercury compounds.

2.2 Specified Industry Subject to Guidance and Average Wastewater Quality Classified by Industry-by-industry

The public sewer users that possibly discharge wastewater in excess of the effluent limitations set forth by Law or Ordinance are specified as industry or establishment required for guidance. The subject industry is divided into 21 categories. Table 2.2 illustrates an average wastewater quality discharged from major type of industries.

Table 2.2 Average effluent qualities for industry-by-industry categories

(mg/ℓ)

	Chromium	Copper	Zinc	Cadmium	Lead	Arsenic	Total mercury	SS
Foods	–	–	–	–	–	–	–	31
Textile Mills	0.008	–	0.06	–	0.05	–	–	96
Pulp & paper	0.01	–	0.17	–	0.05	–	–	121
Iron & steel	0.02	0.34	2.61	–	0.003	–	–	15
Metals	0.31	2.48	4.22	0.001	0.27	–	0.000005	18
Machinery-1	0.44	0.41	0.42	0.0007	0.03	–	0.00009	7
Machinery-2	0.10	2.26	0.83	–	0.009	–	0.0002	4
Electroplating	4.40	9.37	13.4	0.0008	0.06	–	–	18
Printing	0.77	2.51	7.02	0.0009	0.05	–	–	1
Chemicals	0.06	0.06	2.99	0.0004	0.17	–	0.00002	17
Leather tanning & finishing	12.4	–	–	–	–	–	–	270
Ceramics	0.13	0.007	0.17	0.11	0.13	–	–	32
Wastes disposal	–	–	0.002	–	–	–	–	–
Others	0.0008	0.003	0.12	0.00008	0.06	0.000004	0.000002	–

2.3 Situation of Pretreatment Facility Installed

As seen in Table 2.3, the ratio of the pretreatment installed is reaching a higher percentage year by year. Recent status of the pretreatment facility installation of industry-by-industry categories is shown in Table 2.4.

Table 2.3 Status of the pretreatment installation

Fiscal year	1975	1976	1977	1978	1979 (as of end of December)
Specific industrial categories	2,789	3,043	3,946	4,269	4,634
No. of installation completed in the year	439	419	857	455	227
Cumulative No. of installation completed	1,940	2,359	3,216	3,671	3,898
Installation ratio (%)	69	78	82	86	~~84~~ 91%

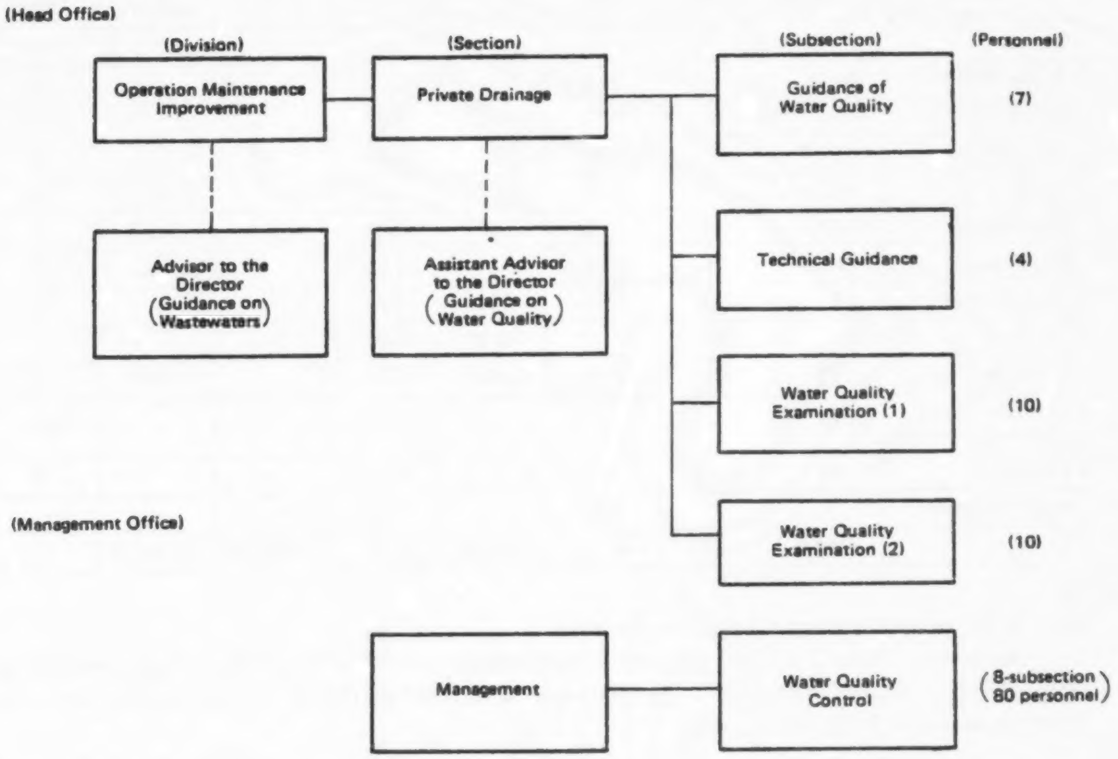
for FY

2.4 Surveillance and Guidance Program for Industrial Wastewater

2.4.1 Structure of execution

The surveillance for the industrial wastewater and guidance for the pretreatment facility installation and maintenance are executed by the organization as shown in Table 2.5.

Table 2.5 Organization chart for implementation of surveillance and guidance program related to industrial wastewaters


(Head Office)
(Division)
(Section)
(Subsection)
(Personnel)
Operation Maintenance Improvement
Private Drainage
Guidance of Water Quality
(7)
Advisor to the Director (Guidance on Wastewaters)
Assistant Advisor to the Director (Guidance on Water Quality)
Technical Guidance
(4)
Water Quality Examination (1)
(10)
(Management Office)
Water Quality Examination (2)
(10)
Management
Water Quality Control
(8-subsection 80 personnel)

(Note) Dotted line denotes the staff.

Total 113

Overall operation is shared at the head office. The subsection in charge of guidance on wastewater quality has an assignment to give a guidance to the installation and operation of the pretreatment facility, and to give a guidance and control of the implementation of the regulation related to the industrial wastewater quality. The subsection in charge of technical guidance deals with an investigation and guidance in the technology related to the pretreatment of the industrial wastewater. The subsection of water quality examination conducts the examination and test of wastewater quality discharged from industry.

Direct surveillance and guidance to the factory are conducted by the subsection of water quality control at the Management Office. In other words, the assignment of the office includes an examination of wastewater quality, an acceptance of application, a guidance to the pretreatment facility installation and maintenance, and regulating industrial wastewater quality.

2.4.2 Status of surveillance

Frequency of surveillance conducted was, on average, 1.4 times per one factory as seen in Table 2.6. This may not be sufficient. However, a priority of on-the-spot inspection was given to the factories that discharge harmful pollutants such as heavy metals, the factories that discharge wastewater of more than 100 m^3/day, and the factories discharge the regulated substances such as acidic wastes, copper and zinc. The 50 factories received on-the-spot inspection of more than five times a year. The cases of violation pointed out by the surveillance program are shown in Table 2.7.

Table 2.6 Annual number of on-the-spot inspection per one industry

(FY 1979)

Factory subject to surveillance	Aggregated number of on-the-spot inspection	Average
4,269	5,931	1.4

Table 2.7 Status of violation

Fiscal year	No. of case for surveillance	No. of case in violation	Ratio of violation (%)
1976	3987	1716	43
1977	6025	2283	38
1978	6198	2050	33
1979 (as of December)	5162	1451	28

No. of violation cited above includes cases of minor violation.

2.4.3 Steps for regulation of industrial wastewater

The factory in violation will receive a phase-by-phase administrative guidance and an administrative punishment. The phase-by-phase guidance consists of oral notice, written notice, warning, hearing and order for improvement including order for suspension of wastewater discharge.. The state of violation is shown in Table 2.8.

Table 2.8 Trend of control under regulations

Fiscal Year	Notice	Warning	Hearing	Order for improvement
1976	1368	348	31	26
1977	1921	404	21	13
1978	1354	368	9	35
1979 (As of December)	1150	257	13	14

2.4.4 System for nomination of person responsible for wastewater quality control

In connection with the surveillance program for wastewater, the system for nomination of the person responsible for wastewater quality control was established in April 1978 under the Sewerage Ordinance of Tokyo Metropolitan Government. The purpose of such promulgation is to clarify where the responsibility lies in the inner side of the factories or industries and to effectuate a maintenance and control of the pretreatment facility including the treatment facility for sewage discharged from the specific facilities.

The job description of the person responsible for wastewater quality control is:

a. proper management of how-to-use facility which produces sewage, the volume of sewage and wastewater quality;

b. maintenance and control of the pretreatment facility, record of operation log, and taking other necessary measures;

c. measurement and record of wastewater quantity and quality to be discharged to the public sewer;

d. data related to the sludge produced at the pretreatment facility; and

e. measures in case of the facility failure and emergency.

The qualification of the person responsible for wastewater quality control shall be the person who holds the qualification for a supervisor of pollution control under the Law concerning Consolidation of Organization for Pollution Control at Specified Factory, or the person who is qualified for a supervisor of pollution control under the Ordinance for Pollution Control of Tokyo Metropolitan Government, and the person who completed lessons at a short course held by the General Manager who is managing the sewerage system of Tokyo.

The industry categories requiring for guidance must select the person responsible for wastewater quality control and must be filed with the Sewage Works Bureau of Tokyo. Number of the person responsible for wastewater quality control is shown in Table 2.9.

Table 2.9 No. of the person responsible for wastewater quality control

	Specific factories	No. of the person selected	Ratio of selection (%)
1979 (as of end of March)	4,269	2,833	66.4
1979 (as of end of December)	4,634	3,469	74.9

An independent management at each industry is gradually strengthened by an establishment of the system for placing the person responsible for wastewater quality control, thus resulting in the tendency of reducing number of cases in violation.

3. FINDING AND GUIDANCE TO THE INDUSTRY NOT INSTALLED THE PRETREATMENT FACILITY

The percentage of the pretreatment facility installed is on increase year by year. Total number of industry categories subject to guidance is 4,629, out of which the number of industry that has not installed the pretreatment facility is 497 as of the end of March, 1979.

Table 3.1 Contrary trend to the pretreatment installation

Classification / Reason	Minor-scale industry	Small- and medium-scale industry	Large-scale industry	Total
Financial difficulty	109	155	1	265
Difficulty in spacing	28	5	2	35
Planning to move	10	1	0	11
Planning to install	61	6	13	80
Subject to surveillance	83	24	54	161
Total	291	191	70	552

Most of them are minor- or small- and medium-sized industries. For reference, a breakdown of the reason for those who have not installed the pretreatment facility is presented in Table 3.1, which is the result of the investigation made at the end of 1977. It is presumed such a tendency has been continuing without a significant change.

Viewing from enterprise-size wise, the industry referred to above comprises 53% of minor-sized industry operated with not more than 20 employees, 34% of small- and medium-sized industry operated with not more than 100 employees and paid-in capital of not more than 10 million yen, and 13% of large-sized industry. The ratio of minor-sized industry is more than a half. "Planning" described in Table 3.1 includes the industry that have already contracted with manufacturer to install the pretreatment.

Though the industry has an intention of installing the pretreatment, those who still have no prospect of installation by the reasons of "financial difficulties" or "difficulty in securing land" will comprise 64% of substantial industry that has not installed the pretreatment (472 factories excluding those in the stage of planning). The number of minor-sized industry comprises a half of the whole.

"Subject to surveillance" represents the industry required for continuous surveillance to guide the installation of the pretreatment facility. This classification comprises 34% of the substantial industry that has not installed the pretreatment. Tokyo Metropolitan Government will strongly promote the guidance for installation of the pretreatment in a manner of giving a full explanation regarding the treatment process, the financing aids (loan system) and others, basing on the results obtained from the study of the industry categories, the enterprise-size wise and the reasons for not having installed the pretreatment yet.

4. LOAN SYSTEM

Those industries intending to install new pretreatment, or to enlarge or modify the existing plant, or improve the production process can utilize the financial aids such as loan system of governmental agencies. The outline and the status of loan utilization are shown in Table 4.1.

Table 4.1 Financing mediation program for pretreatment installation and its status of utilization

Bodies for mediation	Name of financing aid system	Terms and conditions			Fiscal year of 1977		Fiscal year of 1978		Fiscal year of 1979 (•2)	
		Limit of loan (Yen)	Annual interest	Term of redemption	No. of case	Amount (Yen)	No. of case	Amount (Yen)	No. of case	Amount (Yen)
Pollution Control Bureau, Tokyo Metropolitan Government	Finance mediation system for pollution control fund of Tokyo Metropolitan Government	20 million	2% for industry with 10 employees or less 4% for industry with 11 employees or over	Within 7 years	74	(million) 501.27	21	(million) 139.60	6	(million) 38.18
Pollution Control Bureau, Tokyo Metropolitan Government	Loan system of pollution control fund of Tokyo Metropolitan Government	7 million	2% for industry with 10 employees or less 4% for industry with 11 employees or over	Within 8 years	0	0	0	0	1	1.50
Labor and Economic Bureau, Tokyo Metropolitan Government	Fund loan system for equipment modernization of small- and medium-scale industry	12 million	No interest	Within 12 years	0	0	0	0	0	0
Specified Ward of Tokyo Metropolitan Government (•1)	Loan system for pollution control fund	3 million	2%	Within 5 years	81	212.50	63	179.50	34	98.76
Environmental Pollution Control Service Corporation	Loan system for pollution control facilities	Within 80% of construction cost for small- and medium-scale industry and within 50% for large-scale industry	6.5 % 7.35%	Within 10 years	0	0	1	100.00	1	80.00
Smaller Enterprise Finance Corporation	Loan system for industrial pollution control	210 million	First 3 years: 7.15% Remaining: 7.65%	Within 10 years	0	0	4	41.50	7	151.01
Peoples' Finance Corporation	Loan system for pollution control	25 million	First 3 years: 7.15% Remaining: 7.65%	Within 10 years	27	101.90	12	49.58	9	28.15
Environmental Sanitation Finance Corporation	Loan system for treatment facility of polluted water	22 million (additional 15 million yen is allowable)	First 3 years: 7.15% Remaining: 7.65%	Within 10 years	0	0	0	0	0	0
TOTAL					182	815.67	101	510.18	58	397.60

(Note) (•1) denotes the system being employed in Bunkyo Ward and presented as an example
(•2) represents the status as of end of December 1979

5. MAINTENANCE OF THE PRETREATMENT FACILITY

Even in the industries that have already installed the pretreatment may sometimes violate the effluent standards if the facility is not maintained or controlled properly. As a cause of violation, it can be broadly classified into two categories. One is that a problem lies in the pretreatment itself, and another is that a problem lies in the operation and maintenance of the pretreatment facility.

As seen in the statics shown in Table 5.1, the latter comprises about a half. Improper control of instrumentations and improper chemical groupings of wastewater are seen remarkably. The Sewage Works Bureau of Tokyo Metropolitan Government imposed, as stated earlier, an obligation on the industrial categories to select the person responsible for wastewater quality control with the aims to have him control and maintain properly the pretreatment facility.

Table 5.1 Classification of causes for violation

Cause for violation	Ratio (%)
Constructional defect of pretreatment facility	12
Improper capability of pretreatment facility	10
Improper control of raw water including chemical groupings of effluent	13
Improper control of consumable materials such as water treatment chemicals	12
Improper control of measuring instruments for facility	27
Improper operation of pretreatment facility	19
Discharging wastewater without treatment	7

6. RECENT TREND OF THE PRETREATMENT INSTALLATION

6.1 General Trend

6.1.1 Treatment process

The industry discharging wastewater of less than 1 m^3/day employs a batch treatment and a continuous treatment is practised for wastewater of not less than 1 m^3/day. If the control method is divided into automatic and manual control mode, the continuous treatment employs automatic control while the batch treatment employs automatic and manual control which can be split fifty-fifty.

6.1.2 Utilization of recoveries

The industry develops a tendency to reclaim for reuse of wastewater by recovering partially or wholly. The collection methods employed are ion exchange process, electrodialysis and evaporation process. Another process is to polish up the treated wastewater by using filter after treatment of pressurized flotation or chemical precipitation. Among the processes, the evaporation is employed for the treatment of rinsing water of chromium plating and the chromium (VI) recovered by condensation is returned to the plating bath for reuse. Reclaimed water by the ion exchange process is collected, but the resin regeneration wastewater is discharged into the sewer after treated chemically.

The electrodialysis is used for recovery of copper, chromium, etc., but the operation is carried out at the most effective electrolytic concentrations. The residual concentrations are still high so that the wastewater is required to be treated chemically prior to discharging.

6.1.3 Treatment of copper and zinc

The regulation for copper and zinc was also imposed on the industry discharging wastewater of less than 50 m^3/day in May, 1975. The treatment of zinc was a matter of concern, but it was proved that the zinc was eliminated by a hydroxide coagulation. Example of flow diagram is shown in Figure 6.1.

When copper and zinc group wastewaters contain cyanides and chromium (VI), the wastewater is divided into each group, but, if not, the wastewater is treated by the chemical precipitation process in the group of acid-alkali. Example of flow diagram for batch treatment and for continuous treatment are shown in Figures 6.2 and 6.3, respectively.

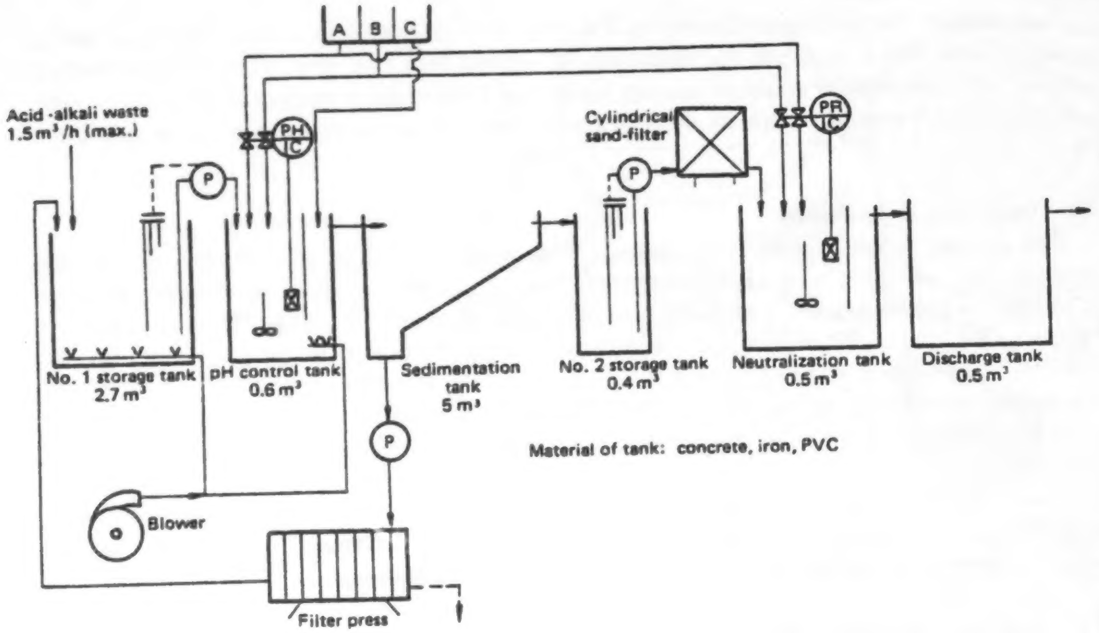
6.1.4 Prepackaged unit treatment

Most of unit type treatment made of steel and PVC are in prepackaged unit that is fully equipped with necessary treatment process. The prepackaged unit requiring only water and electrical connections on-site operation tends to be employed at an experimental facility or a research laboratory where the volume of wastewater is less than 0.5 m^3/day and many substances are required to be treated. Metal industry and machinery manufacturing with an operation of plating process are also in tendency of employing such a prepackaged unit type treatment.

6.1.5 Dewatering of sludge

Most of industries that produce the sludge as a result of wastewater treatment tend to install the dewatering device. This is to aim at reducing volume to be disposed of finally. Small-sized filter press and bag filter press are employed mostly at the industry which produces less volume of sludge. Centrifuge, vacuum dehydrator, belt press and filter are employed in other area.

Fig. 6.1 Flow diagram for treatment of pH, SS, Cu, Zn at N Metals
Capacity: 9 m³ /day


A
B
C
Acid -alkali waste
1.5 m³ /h (max.)
P
PH
IC
Cylindrical sand-filter
P
PR
IC
No. 1 storage tank
2.7 m³
pH control tank
0.6 m³
Sedimentation tank
5 m³
No. 2 storage tank
0.4 m³
Neutralization tank
0.5 m³
Discharge tank
0.5 m³
P
Blower
Filter press

Material of tank: concrete, iron, PVC

Fig. 6.2 Flow diagram for treatment of pH, CN, Cr $^{6+}$, T-Cr, Zn at T metal-plating
Process: batch (manual operation)
Capacity: 0.6 m³ /day
Material of tank: Iron, PVC

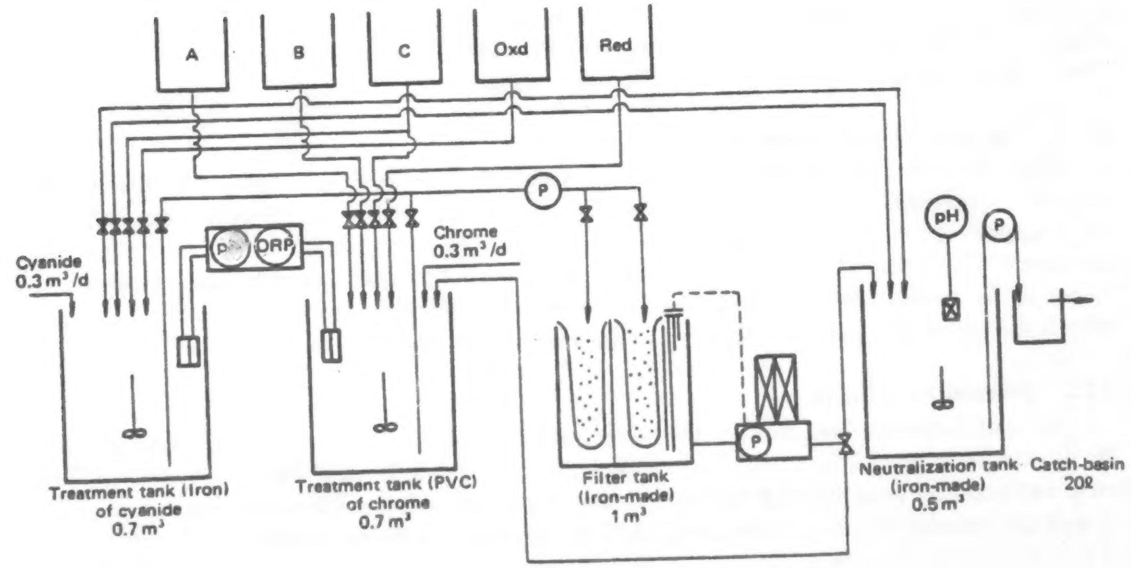
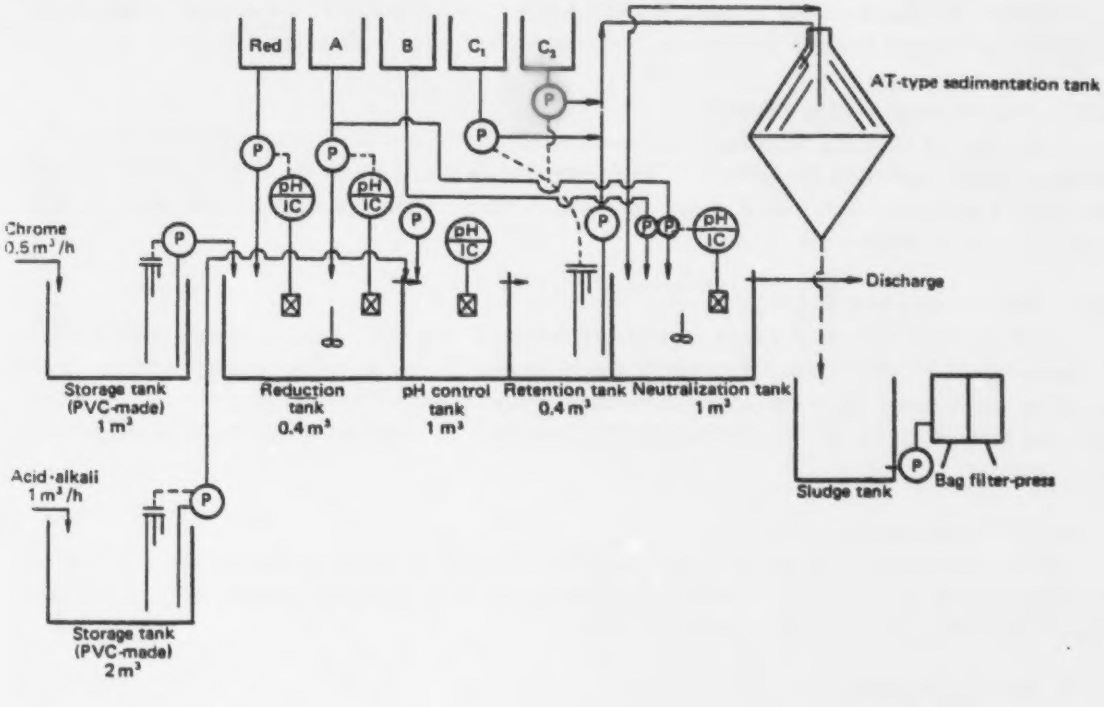

A
B
C
Oxd
Red
P
pH
ORP
Cyanide
0.3 m³ /d
Chrome
0.3 m³ /d
pH
P
P
Treatment tank (Iron)
of cyanide
0.7 m³
Treatment tank (PVC)
of chrome
0.7 m³
Filter tank
(Iron-made)
1 m³
Neutralization tank
(iron-made)
0.5 m³
Catch-basin
20ℓ

Fig. 6.3 Flow diagram for treatment of zinc wastewater containing cyanide and chromium (VI) at S Metal-plating
Substances to be treated: pH, Cr^{6+}, T-Cr, Zn at S plating
Process: Automatic and continuous operation
Capacity: 12 m^3/day


Red
A
B
C_1
C_2
P
pH IC
AT-type sedimentation tank
Discharge
Chrome 0.5 m³/h
Storage tank (PVC-made) 1 m³
Reduction tank 0.4 m³
pH control tank 1 m³
Retention tank 0.4 m³
Neutralization tank 1 m³
Sludge tank
Bag filter-press
Acid-alkali 1 m³/h
Storage tank (PVC-made) 2 m³

6.2 Features of Treatment Classified by Industry-by-Industry

6.2.1 Foods processing industry

The items of wastewater to be treated are pH, BOD, SS and animal/vegetable oils. Generally, volume of wastewater for treatment is large.

The treatment process by activated sludge, rotating biological contactor, contact aerator, trickling filter, coagulation and sedimentation and pressurized flotation are employed. Filter press and belt press are used for dewatering of sludge.

6.2.2 Textile industry

Most of wastewaters in the process are dyeing wastes and the item to be treated is pH. Since a large volume of wastewater are discharged, the continuous and automatic control system are employed. Besides neutralization, reduction, chemical precipitation and dewatering plant are equipped with where chromic acid is used in the process.

6.2.3 Metal products manufacturing

The items to be treated are pH, copper, zinc and SS. In any event, pH is always controlled. In some cases, the sedimentation process is employed additionally.

6.2.4 Machinery manufacturing (1)—(with metal-plating process)

Wastewater treatment is required as same level as the regular metal-plating industry, since the metal-plating is included in the manufacturing process. The objective items for treatment are pH, SS, Cr^{6+}, total Cr, CN, Cu, Zn, Fe, etc.

6.2.5 Machinery manufacturing (2)—(without metal-plating process)

Though the metal-plating process is not included, the factory, where chrome treatment is employed, is required to equip with the reduction treatment since it discharges Cr^{6+}.

6.2.6 Regular metal-plating industry

This type of industry discharges less wastewater, compared with other industry categories. Mostly a batch treatment is employed. The treatment facility such as the reduction process and oxidation is generally used, but, in some cases, ion exchange resin and chromic acid are recovered by evaporation to condensate.

6.2.7 Plate making and printing

Most of factories are required to treat pH only. If the plate making process includes the development of PS plate (aluminum-made), the neutralization by injecting carbonic acid gas is required in many cases since the alkali wastewater is discharged. Where etching process of copper and zinc plate is employed, the wastewater is neutralized by addition of caustic soda for further sedimentation.

6.2.8 Chemical industry

Wastewater quality discharged from industrial factories is varied, so the treatment process is not standardized. In general, the raw materials are recovered to the most possible level. In the most cases, the objective item for treatment is pH only.

6.2.9 Ceramics industry

When the zinc is discharged in the process of glass product manufacturing, the wastewater is treated by a hydroxide coagulation. Fluorine is precipitated in a form of CaF_2 by a lime coagulation.

6.2.10 Others

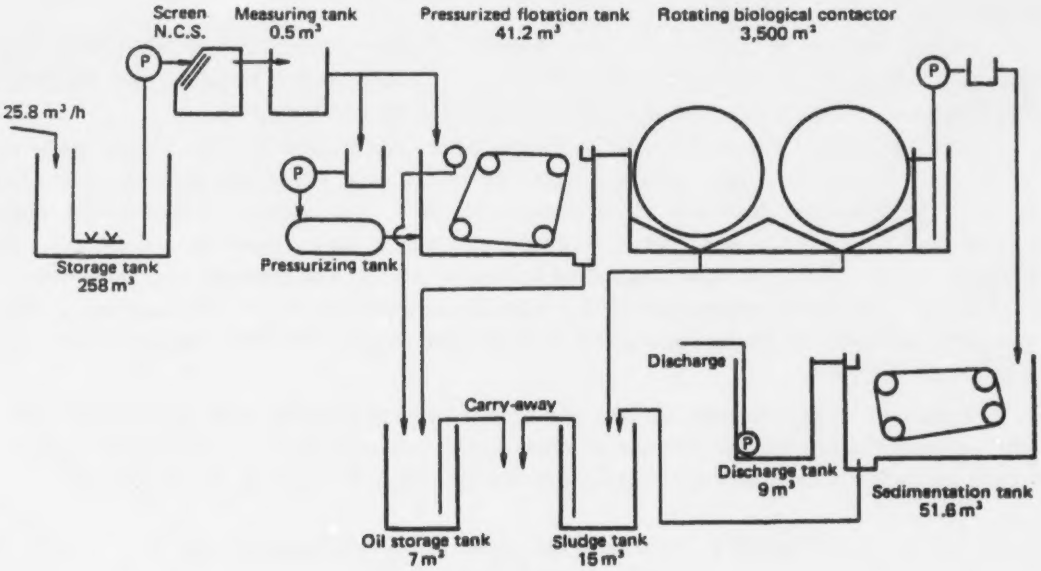
a. School, institute, establishment for chemical analysis, laboratories

Establishment in this category is featured that it discharges a small amount of wastewater but contains various harmful substances. In most cases, a prepackaged unit type wastewater treatment process for laboratory is used. Operation mode of the unit is a batch treatment by manual or semi-automatic control. Among heavy metals, Hg and Cd are treated checmically and a chelate resin or activated carbon adsorption is installed to prevent a leakage of Hg and Cd.

b. Hotel and lodging

Wastewater contains wastes from a cuisine, thus resulting in a large amount of oil contents. Flotation process is employed in the prior or main stage, and then a rotating biological contactor, activated sludge process and coagulation process are employed for removing BOD and SS in the later stage. An example of flow diagram which combines the pressurized flotation and rotating biological contactor for the treatment of wastewater discharged from O Hotel is presented in Figure 6.4.

Fig. 6.4 Flow diagram for treatment of cuisine wastewater at O Hotel
Substance to be treated: BOD, SS, Oil
Process: Pressurized flotation and rotating biological contactor
Capacity: 516 m^3/day
Tank: Concrete-made


Screen
N.C.S.
Measuring tank
0.5 m³
Pressurized flotation tank
41.2 m³
Rotating biological contactor
3,500 m²
P
25.8 m³/h
Storage tank
258 m³
P
Pressurizing tank
P
Discharge
Carry-away
P
Discharge tank
9 m³
Sedimentation tank
51.6 m³
Oil storage tank
7 m³
Sludge tank
15 m³

6.3 Example of a Joint Treatment of Wastewater Discharged from Collective Minor-Sized Factories

In general, the minor-sized factories are located in the mixed area of housing and factories. Nearby inhabitants have voiced their desire that the government would buckle down to the problem of pollution control. However, those factories are facing with a certain difficulty in installing the effective pretreatment facility due to smaller land space of the factory and financial difficulties. Also, since those factories employ only limited number of employees, a difficulty in placing the persons who maintain and control the wastewater treatment plant on full-time duty has been involved. In such circumstances, the factories in this category are apt to pay less attention to proper treatment. In order to resolve such a bottleneck, it will be essential for the government to promote actively the enforcement of a policy. The measures to be taken are to induce a relocation of the factory located in the mixed area of housing and industry, to have them accelerate the improvement of production process aiming at pollution prevention, and at the same time, to have them install a joint treatment facility for wastewater.

Jyonan Denka Kyodo Kumiai (Jyonan Galvanization Cooperative Association) promoted to collectivize metal-plating factories and to install a joint treatment facility of wastewater (Metal-plating Center). The scheme was planned on the grounds mentioned previously and brought into existance as a series of "promotions of pollution abatement of electroplating industry and for joint treatment". Metal-plating Center was organized and the wastewater treatment was completed in March 1975 and put on stream in September 1975. Further information is given in the following chapter.

6.3.1 Structure of facility

"Metal-plating Center", comprizing of collective 10 factories, has a land space of 4,600 m^2 and its building floorage of 2,300 m^2, out of which 150 m^2 is allotted for a joint wastewater treatment facility, exhaust gas scrubber and analysis room. Business status is classified into 7 copper-plating, 3 zinc-plating, 7 chrome-plating, 7 nickel-plating, 1 tin-plating and 3 silver-plating.

6.3.2 Joint treatment facility of wastewater

Overall wastewater discharged from individual factory amounts to about 170 m^3/day. Independent treatment facility for individual group of wastewater is installed, and segregation of wastewater depending on group and separated treatment are well managed.

Group of wastewater is divided into 10 groupings, that is, zinc cyanide, copper cyanide, acid-alkali iron, acid-alkali copper, nickel, copper pyrophosphate, chromate, chrome plating, highly-concentrate acid-alkali iron and highly-concentrate acid-alkali copper. Fundamental treatment processes employed are an oxidation by sodium hypochlorite for cyanides, the reduction by sodium bisulfite for chromium (VI) and coagulation for heavy metals. The treatment of highly-concentrate cyanide wastewater has been entrusted to Jyonan Treatment Center (refer to Chapter 6.4—Example of a joint treatment of highly-concentrate cyanide wastewater). The flow diagram of the treatment is shown in Table 6.5.

Sludges of nickel, chrome, copper and zinc group are dewatered by a filter press, and then sent to the refineries for the purpose of cyclic use of raw materials. The dewatered sludge transported annually and content concentrations of effective ingredients are shown in Table 6.1.

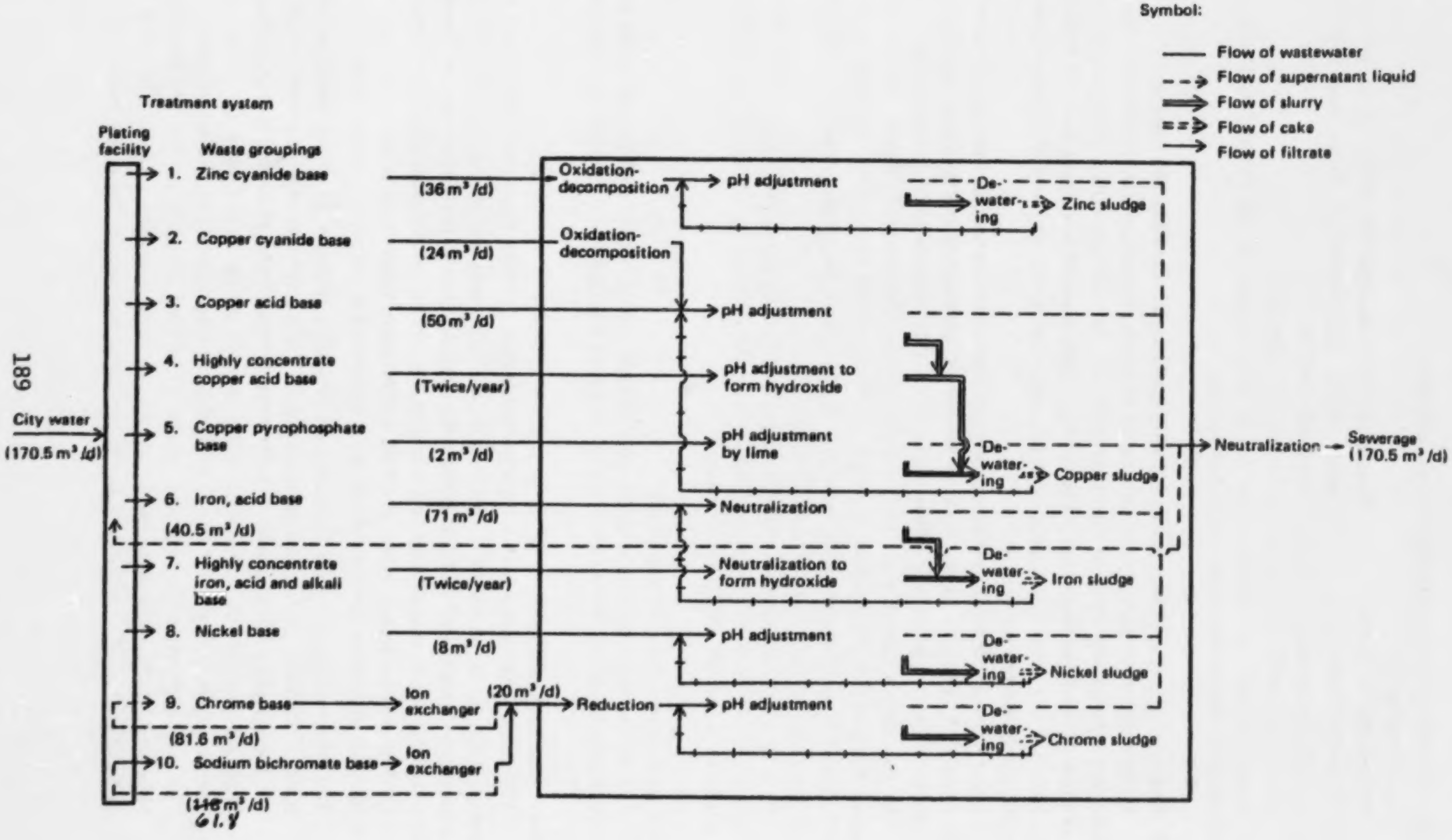
Table 6.1 Volume of dewatered sludge and concentration of effective consituent content

Type of dewatered sludge	Weight transported annually (ton/year)	Effective constituent content (%)
Chromium group	44	22
Copper group	24	31
Zinc group	15	25
Iron group	62	5

(Note) Volume of discharge is expressed in wet weight.

1. What punitive measures are used to gain compliance when hearings and notices are not effective.

Fig. 6.5 Flow diagram of a joint-treatment in metal-plating center


Symbol:
Flow of wastewater
Flow of supernatant liquid
Flow of slurry
Flow of cake
Flow of filtrate
Treatment system
Plating facility
Waste groupings
1. Zinc cyanide base
2. Copper cyanide base
3. Copper acid base
4. Highly concentrate copper acid base
5. Copper pyrophosphate base
6. Iron, acid base
7. Highly concentrate iron, acid and alkali base
8. Nickel base
9. Chrome base
10. Sodium bichromate base
City water (170.5 m³/d)
(40.5 m³/d)
(81.6 m³/d)
~~(116 m³/d)~~ 61.8
(36 m³/d)
(24 m³/d)
(50 m³/d)
(Twice/year)
(2 m³/d)
(71 m³/d)
(Twice/year)
(8 m³/d)
Ion exchanger
Ion exchanger
(20 m³/d)
Oxidation-decomposition
Oxidation-decomposition
Reduction
pH adjustment
pH adjustment
pH adjustment to form hydroxide
pH adjustment by lime
Neutralization
Neutralization to form hydroxide
pH adjustment
pH adjustment
De-watering
Zinc sludge
Copper sludge
Iron sludge
Nickel sludge
Chrome sludge
Neutralization
Sewerage (170.5 m³/d)

6.3.3 Recycling for reuse of industrial wastewater

a. Ion exchange

Wastewater of chromate and chrome plating is treated individually by an ion exchanger, and most of treated water is reused for rinsing water. The wastewater of ion exchange resin regeneration is treated separately at the joint treatment facility.

b. Treated water of joint treatment facility of wastewater

About 20% of the treated water ($40\,m^3$/day) is sent to the factory, and reused as roughing wash water in the pretreatment process of plating.

6.4 Example of a Joint Treatment of Highly-Concentrate Cyanide Wastewater

Wastewater of cyanide group generated at an electroplating can be broadly classified into the wastewater containing cyanide concentrations of 50–200 mg/ℓ that is discharged normally and continuously, and the wastewater containing highly-concentrate cyanide of 15,000–45,000 mg/ℓ. It is a matter of course to treat the normally discharging wastewater individually at the factory that produces such wastewater.

However, since the highly-concentrate wastewater contains high concentrations of free cyanide and complex cyanide, a higher level of treatment technology and a higher cost for treatment are required. Collective treatment will be more acceptable economically or technically.

In the collaboration with Tokyo Metropolitan Government, Tokyo Metal-plating Pollution Control Corporative Association (formed by membership of 1,100 companies) built a joint treatment facility named as Jyonan Treatment Center, which has a land space of $1{,}300\,m^2$ and wastewater treatment capacity of 200 kiloliters/month.

In June 1972, the Center began to collect the highly-concentrate cyanide wastewater from the metal-plating factories in Tokyo area and is now functioning to transport the treated sludges, after an intermediate treatment, for reclamation.

6.4.1 Objective items for treatment

The wastewater containing highly-concentrate cyanide of electrolytic degreased liquid, exhaust neutralization liquid of plating bath, nickel-stripping liquid is the objective items for treatment.

6.4.2 Treatment method

The treatment method of a highly-concentrate cyanide wastewater employed in the Jyonan Treatment Center is, so called, a steam dryer and high temperature incineration. This method is to evaporate the wastewater to form dried solids, which receive heat treatment at the metal refinery where the cyanide is decomposed and heavy metals are recovered as an effective ingredient.

Outline of the steam dryer and high temperature incinerator is shown in Figure 6.6 and the flow diagram of Jyonan Treatment Center is illustrated in Figure 6.7, respectively.

a. The wastewater collected from the metal-plating industry by three trucks is maintained in a state of alkaline, and stored temporarily in the wastewater receiving tank which is served also for precipitation tank as well. The volume of wastewater collected is ranging from 140 to 210 kiloliters per month. The wastewater is kept gently in the tank for about 48 hours, and separated into supernatant and settled sludge while allowing settlement of solids.

b. The supernatant is condensated under a reduced pressure in the condensation tank, and reformed to the flake state by drying on the surface of drum dryer. The dried solids in form of flake of 30–35 tons per month are produced and sent to the metal refinery in container.

c. The dried solids are fed into a high temperature furnace of the refinery and incinerated at about 1,200°C for two hours to decompose cyanide as well as to recover zinc and copper in the incineration stage. Waste gas, cleaning waste and residual suspended solids are turned to non-pollutant substances.

d. Cooled condensation discharged from the condensation tank and drum dryer contains ammonia which is produced in a reaction of $[NaCN + 2H_2O \rightarrow HCOONa + NH_3]$. Therefore, the cooled condensation is sent to an ammonia stripper where the ammonia stripping is achieved.

e. Ammonia-free wastewater discharged from the ammonia stripper contains a small amount of cyanides and heavy metals. The wastewater will be treated by alkali treatment to remove cyanides and then by sedimentation to eliminate heavy metals. The settled hydroxide is sent to the metal refinery together with the dried solids after dewatering.

f. The treated water is discharged into the sewer after adjusting pH.

Fig. 6.6 Outline of steam dryer and high-temperature incinerator

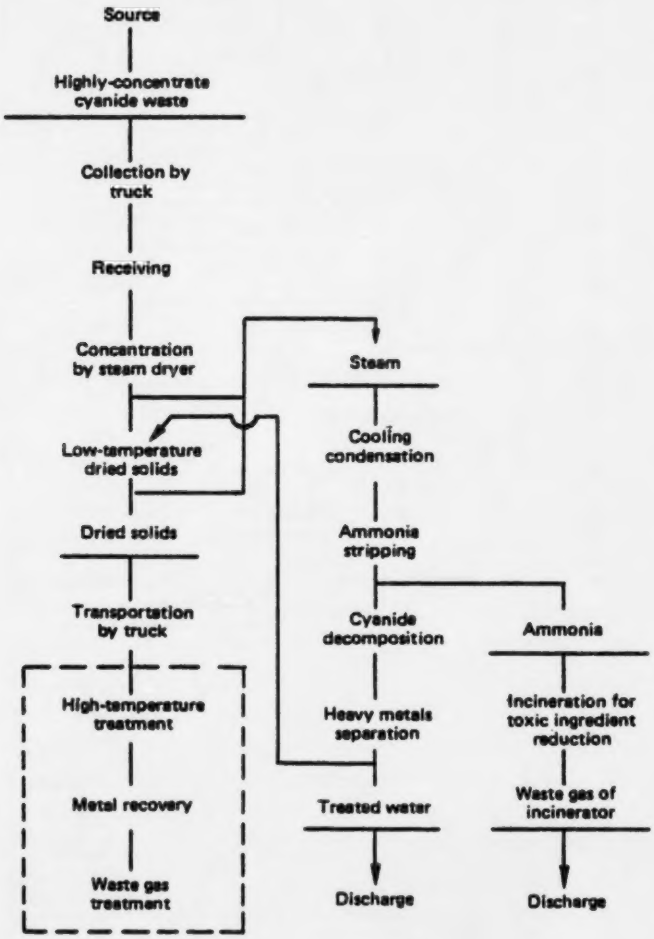
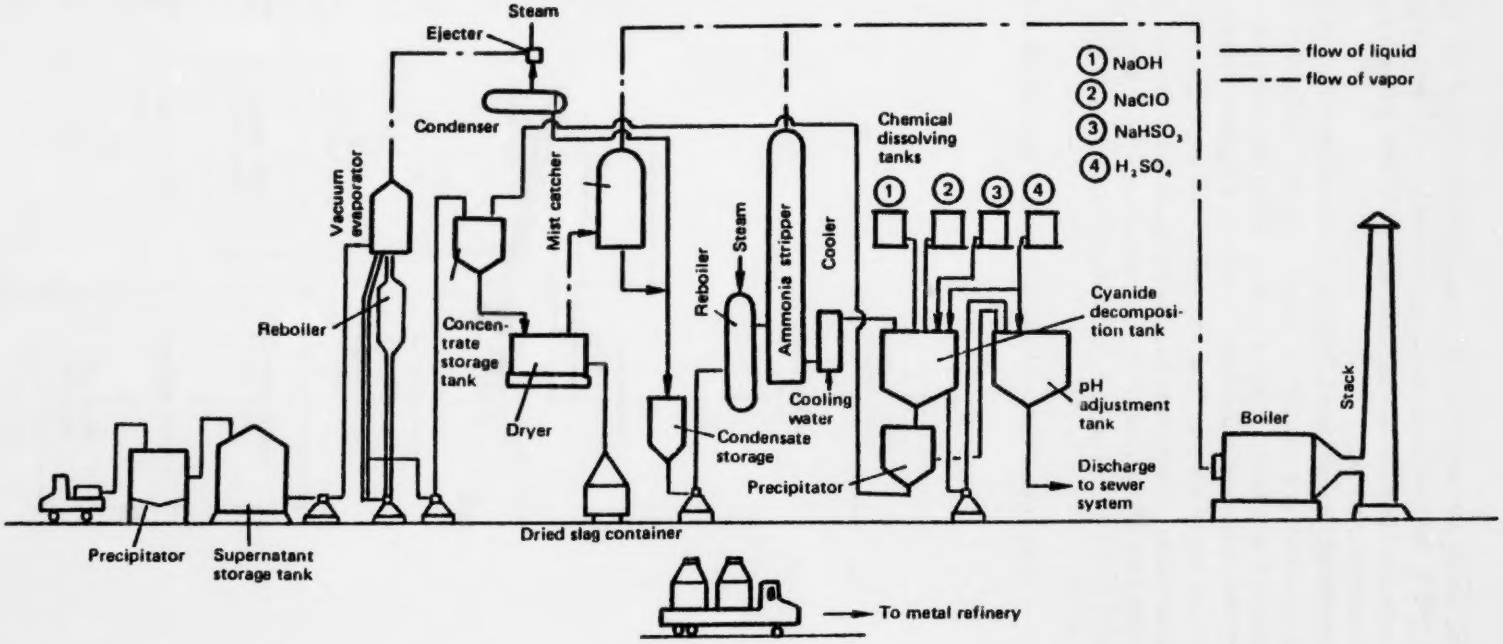

Source
Highly-concentrate cyanide waste
Collection by truck
Receiving
Concentration by steam dryer
Steam
Cooling condensation
Low-temperature dried solids
Dried solids
Ammonia stripping
Transportation by truck
Cyanide decomposition
Ammonia
High-temperature treatment
Heavy metals separation
Incineration for toxic ingredient reduction
Metal recovery
Treated water
Waste gas of incinerator
Waste gas treatment
Discharge
Discharge

Fig. 6.7 Flow diagram of treatment plant at Jyonan Treatment Center


Steam
Ejecter
Condenser
Vacuum evaporator
Reboiler
Mist catcher
Concen-trate storage tank
Dryer
Dried slag container
Reboiler
Steam
Ammonia stripper
Cooler
Cooling water
Condensate storage
Precipitator
Chemical dissolving tanks
① NaOH
② NaClO
③ NaHSO₃
④ H₂SO₄
① ② ③ ④
Cyanide decomposi-tion tank
pH adjustment tank
Discharge to sewer system
flow of liquid
flow of vapor
Boiler
Stack
Precipitator
Supernatant storage tank
To metal refinery

6.4.3 Status of treatment

Analytical value of ingredients of each process at the Jyonan Treatment Center is presented in Table 6.2.

Table 6.2 Chemical composition of each process at Jyonan Treatment Center

Sample Item	Highly concentrate wastewater	Effluent to sewerage	Dried solids
Unit	mg/ℓ	mg/ℓ	%
Total cyanide	21,000	0.7	15
Cadmium	50	0.02	0.4
Copper	5,500	1.1	6
Zinc	3,500	0.5	5
Total Fe	400	1.1	0.8
Total chromium	50	0.08	0.05
Nickel	1,400	0.2	0.9

[Reference Materials]

1. Letter requesting for investigation on wastewater
2. Inquiry sheet for industrial wastewater
3. Ledger of factory subject to guidance

1. Letter requesting for investigation on wastewater

To:

From: Sewage Works Bureau
Tokyo Metropolitan Government

Subject: Request for investigation on industrial wastewater

Recently, an increasing concern on water pollution has been recognized, and an enforcement of legislation concerning Water Pollution Control Law and Sewerage Law has been strengthened gradually.

Now, the Sewage Works Bureau of Tokyo Metropolitan Government has decided to conduct a study of wastewaters discharged from industries.

You are therefore requested that you may necessarily return the enclosed "Investigation sheet on industrial wastewater" to the address indicated at the bottom of this letter not later than (day) (month) (year). Filling-in the blanks may be referred to the specimen attached.

Filled-in form will be returned to:

Any doubt or inquiry may be made to the address above.

Manager of Administration Office

(Note) Production process chart and floor plan are also attached.

2. Inquiry sheet for industrial wastewater

SECRET

Inquiry sheet for industrial wastewater

Date

Name of factory		Representative	
		Person in charge	
Address	Tel:		
Type of business		Major products and amount of production (per month)	
Capital ¥ Area of factory m^2 Employee Working hour from to Holidays			
Wastes required for treatment group m^3/d group m^3/d group m^3/d group m^3/d Wastes not required for treatment m^3/d		Amount of raw materials and chemicals used (per month)	Fluctuation with time of wastewater Continuously Intermittently
Water source and volume for use City water m^3/d Industrial water m^3/d Well water m^3/d River water m^3/d Others m^3/d Total m^3/d		Remarks	

(Note) Production process chart and floor plan are attached.

3. Ledger of factory subject to guidance

Ledger of factory subject to guidance (face)

No.	

Filed on (date)

Name of factory		Representative		Category No.	Receiving wastewater treatment plant
Address	Tel	Person responsible for water quality control	Status		
			Name		

Capital	¥	When plant was on stream	(date)
Area of factory	m²	Major products and amount of production	
Employees			
Working hour	from to		
Holidays			

Required for treatment		Water source and volume for use	
group	m³/d	City water	m³/d
group	m³/d	Industrial water	m³/d
group	m³/d	Well water	m³/d
group	m³/d	River water	m³/d
group	m³/d	Others	m³/d
group	m³/d		
Not required for treatment	m³/d	Total	m³/d

Major raw materials, chemicals and amount consumed
Discharge to POTW m³/d

Objective item of water quality	
Type of specified facility (No., Name)	
Pretreatment facility (installed or not)	

Record of guidance	Date		
Application for use			
Report on water quality improvement			
Application for pretreatment facility			
Application for specified facility			
Completion of pretreatment, etc.			
Application for responsible person for water quality control			
Warning			
Warning			
Warning			
Warning			
Warning			
Hearing			
Order			
Remarks			

Ledger of factory subject to guidance (back)

Production process chart (Flow diagram)

	Sampling (date)	Place of sampling	Items of water quality and measurement						
Raw water									
Checkups of treated effluent at a time of completion									

Floor plan of factory	Histry of major guidance

(Note) Item-by-item punches are provided along margin space to aggregate.

Seventh US/JAPAN Conference
on
Sewage Treatment Technology

DEODORIZATION IN SEWAGE TREATMENT PLANTS OF YOKOHAMA

MAY 1980
TOKYO JAPAN

SHIGEKI MIYAKOSHI
HEAD, CONSTRUCTION DIVITION
SEWAGE WORKS BUREAU
CITY OF YOKOHAMA

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Content

Foreword 201

1. Current State of Deodorization Equipments 202
  - 1.1 Basic Considerations Concerning Odor Control 202
    - 1.1.1 Outline of Legal Regulations Governing Oder 202
    - 1.1.2 Odor Measurement Methods 206
    - 1.1.3 Odor at Sewerage Facilities 208
  - 1.2 History of the Installation of Deodorization Equipment 209
  - 1.3 Actual Results of Operation 214
    - 1.3.1 Measured Values of Odor 214
    - 1.3.2 Operation and Maintenance Costs of Deodorization Equipment 214
  - 1.4 Problems of Existing Deodorization Equipment 214

2. Deodorization Experiments in the Grit Chamber 220
  - 2.1 Purpose 220
  - 2.2 Plan 220
    - 2.2.1 Location of Experiment and its Deodorization Method 220
    - 2.2.2 Target Values of Deodorization 221
    - 2.2.3 Outline of Experiment Equipment 221
    - 2.2.4 Sampling 225
  - 2.3 Profile of Odor 226
    - 2.3.1 State of Sewage Treatment at the Experiment Locations 226
    - 2.3.2 State of Odor Generation at the Grit Chambers 226
  - 2.4 Effects of Deodorization 229
    - 2.4.1 Removal of Odor Unit 229
    - 2.4.2 Removal of Offensive Odor Substances 232
  - 2.5 Summary 238
    - 2.5.1 Original Odor 238
    - 2.5.2 Effect of Deodorization 238
    - 2.5.3 Operation Controlability 239
  - 2.6 Future Problems 239

Afterword 241

Foreword

Improper operation or inadequate equipment at a sewage treatment plant or a pumping station will cause offensive odor. This may lead to complaints from people living in nearby houses.

In recent years, it has therefore become important to control offensive odor when a new treatment facility is constructed.

Odor control is a difficult matter, because sensing the odor is different by individual, and even if a small amount of it is sensed, and there is a fact that many unknown points are still left about the offensive odor substances.

This paper intends to introduce the current state of odor, and its control in Yokohama.

1. Current State of Deodorization Equipments

1.1 Basic Considerations Concerning Odor Control

1.1.1 Outline of Legal Regulations Governing Odor

In Japan, various laws and ordinances have been enacted for the regulation of environmental pollution with the Basic Law for Environmental Pollution Control as the base. Table 1 shows the laws and ordinances regarding offensive odor.

According to the national law and ordinance eight kinds of offensive odor substances are designated at present (Table 2). Their regulation standards are shown in Table 3. These values have been established by the "Six Grades Odor Intensity Method" (Table 4), taking a 2.5 odor grade index as the lower limit and 3.5 as the upper limit.

On the other hand, the actual regulation standards of each local government are determined by the prefectual governor by taking local or special social conditions into account. However, in the large cities designated by Cabinet Ordinance, these standards are to be determined by the mayor. The regulation standards in Yokohama are determined as shown in Table 5. Through a consolidation of these laws and ordinances, the responsibilities of the odor generating source have been clarified and the application of administration treatment made easier.

However, there is a problem as to whether all kinds of offensive odors which exist as combined offensive odor can be controlled by regulating only eight kinds of substances out of dozens of offensive odor substances. Naturally, there are limitations in regulating such substances or an individual basis.

For this reason, it is better considered to detect offensive odor compositely by human sense rather than to regulate individual offensive odor substance for the definite evaluation of the damage caused by offensive odor and the more effective removal.

The measuring method of test by human sense will be described later. Tokyo metropolis has partially revised the Environmental Pollution Control Ordinance and its Operational Regulation in 1977, and is regulating odor unit (Table 6) based upon the "triangle bag test".

Table 1 System of Laws Regarding Offensive Odor

National Laws and Ordinances	Basic Law for Environmental Pollution Control (Law No. 132, 1967)	Article 2	Definition of Offensive Odor
		Article 9	Environmental Standard
		Article 10	Emission Control, etc.
			On the whole, these are general descriptions and the details are covered by the separate Operational Ordinance or Regulations in a more concrete form.
	Offensive Odor Control Law (Law No. 91, 1971)	Article 2	Definition of Offensive Odor Substances (To be established by Cabinet Ordinance.)
		Article 3	Designation of Regulation Area (To be established by Prefectural Governor.)
		Article 4	Regulation Standards (To be established upon the Regulations of Boundary Line of Site and Stack Outlet established by prefectural governors based upon the Order of Prime Minister's Office.)
		Article 6	Public Notice of Designation of Regulated Area
		Article 18	Delegation of Administration Administration within the authority of the prefectural governors authorized under this Law may be delegated to the mayors of municipalities in accordance with the provisions of the Cabinet Order
	(From Article 2 and Article 18 of the Law) Operational Ordinance of Offensive Odor Control Law (Cabinet Order No. 207, 1972)	Article 1	Prescribes 8 substances as offensive odor substances——Refer to Table 2.
		Article 2	Delegation of Administration
		Clause 2	Administration within the authority of the prefectural governors authorized under the Offensive Odor Control Law may be delegated to the mayors of the designated city about the designation of regulated area and the establishment of regulation standards and so on.
	(From Article 4 and Article 6 of the Law) Operational Regulation of Offensive Odor Control Law (Ordinance of the Prime Minister's Office No. 39, 1972)	Article 1	Provides the range of regulation standards for each substance in the boundary line of site. ——Refer to Table 3.
		Article 2	Provides the establishment of regulation standard for each substance at the stack outlet.
		Article 3	Provides that the measuring methods is decided by the Chief of Environment Agency.
Regulations of Yokohama City, etc.	(From Article 2 Clause 2 of the Operational Ordinance of Offensive Odor Control Law) Detailed Operational Regulation of Offensive Odor Control Law (City Regulation No. 90, 1973)		Provides the administration delegated to the mayor based upon the law.
	(From Article 2 Clause 2 of the Operational Ordinance of Offensive Odor Control Law.) Regulated Area and Regulation Standard and so on based upon the Offensive Odor Control Law (City Notice No. 3, 1978)		1. Regulated area (Applied to the urbanized area determined by the Construction Plan of Yokohama International Port City.) 2. Regulation Standard (1) Value of Regulation Standard in the boundary line of site is given in the separate table. ——Refer to Table 5. (2) Regulation Standard at the stack outlet to conform with Article 2 of the Operational Regulation of Offensive Odor Control Law.

Table 2 8 Substances Regulated by the Operational Ordinance of Offensive Odor Control Law

Substance	Regulated Year
Ammonia	1972 by Cabinet Order No. 207
Trimethyl amine	Ditto
Hydrogen sulfide	Ditto
Methyl mercaptan	Ditto
Methyl sulfide	Ditto
Dimethyl disulfide	1976 by Cabinet Order No. 242
Acetaldehyde	Ditto
Styrene	Ditto

Table 3 Regulation Standard Values of the Operational Regulation of Offensive Odor Control Law

(ppm)

Substance	Detective Threshold Value		Recognition Threshold Value	Regulation Standard Values (At boundary line of site)				
Ammonia	0.1	0.3	0.6	1	2	5	10	40
Trimethyl amine	0.0001	0.0004	0.001	0.005	0.02	0.07	0.2	3
Hydrogen sulfide	0.0005	0.002	0.006	0.02	0.06	0.2	0.7	8
Methyl mercaptan	0.0001	0.0003	0.0007	0.002	0.004	0.01	0.03	0.2
Methyl sulfide	0.0001	0.0005	0.002	0.01	0.04	0.2	0.8	20
Dimethyl disulfide	0.0003	0.001	0.003	0.009	0.03	0.1	0.3	3
Acetaldehyde	0.002	0.006	0.01	0.05	0.1	0.5	1	10
Styrene	0.03	0.08	0.2	0.4	0.8	2	4	20
Odor grade index	1	1.5	2	2.5	3	3.5	4	5

(References)

The relationship between the grade index and the each substance is shown as follows:

y = Odor grade index
x = Concentration (ppm)

• Ammonia	$y = 1.6744 \log x + 2.3838$
• Trimethyl amine	$y = 0.9007 \log x + 4.5588$
• Hydrogen sulfide	$y = 0.9502 \log x + 4.1379$
• Methyl mercaptan	$y = 1.2525 \log x + 5.9895$
• Methyl sulfide	$y = 0.7843 \log x + 4.0634$
• Dimethyl disulfide	$y = 1.0 \log x + 4.523$
• Acetaldhyde	$y = 1.053 \log x + 3.948$
• Styrene	$y = 1.4356 \log x + 3.105$

Table 4 Six Grades Odor Intensity Method

Odor grade index	Description
0	No odor
1	Barely detectable odor (Detective threshold)
2	Light odor perceptible of its kind (Recognition threshold)
3	Definite odor
4	Strong odor
5	Overpowering odor

Table 5 Regulation Standard Values at Boundary Line of Site in Yokohama City

(ppm)

Substance	Concentration
Ammonia	1.0
Trimethyl amine	0.005
Hydrogen sulfide	0.02
Methyl mercaptan	0.002
Methyl sulfide	0.01
Dimethyl disulfide	0.009
Acetaldehyde	0.05
Styrene	0.4

Table 6 Regulation Standard Values of Odor Unit in Tokyo Metropolis

Location District	At stack outlet	At boundary line of site
Industrial specialized district. Industrial district.	1000	20
Semi-industrial district Commercial district. Neighborhood commercial district.	500	15
District other than above.	300	10

Note:

1. The odor unit is defined as the air dilution multiple when the air with offensive odor is diluted into the odorless air, and it can be determined by the triangle bag test.
2. When determing the value, the odor unit at the stack outlet must be taken as the averaged value during an operation period, and the odor unit at the boundary line of site must be taken from the instantaneous value at its peak.
3. This regulation must satisfy both values at the stack outlet and the boundary line of site.

1.1.2 Odor Measurement Methods

The odor measurement methods currently used are as follows:

a. Measurement of odor substance concentration by chemical analysis.
b. Sensory test judged by smell.

In Japan, the Offensive Odor Control Law currently designates the use of the chemical analysis method falling under category (a). However in some of the local governments, the sensory test falling under category (b) is used.

Kinds of main odor measurement method are described as follows:

a. Chemical analysis methods

- ① Gas chromato-graphic method
  - FID (Flame Ionization Detector)
    - Hydrocarbons (Styrene)
    - Aldehydes (Acetaldehyde)
    - Amines (Trimethylamine)
  - FPD (Flame Photometoric Detector)
    - Sulfur compounds (Hydrogen sulfide) (Methyl mercaptan) (Methyl sulfide) (Dimethyl disulifde)
- ② Absorptiometric method – Ammonia
- ③ Gas chromatograph mass spectrometory (GC-MS)
- ④ Continuous automatic measurement method
  - Coulometric analysis
  - Gas chromatographic method

b. Sensory test methods

- ① Direct indication method
  - Odor grade index method
    - Five grades
    - Six grades
  - Pleasant or unpleasant indication method
- ② Air dilution method
  - Odorless chamber method
  - Odor bag method
  - Olfactometer method
  - ASTM syringe method

In the investigations and experiments which are noted in this paper, an absorptiometric method was used for ammonia, and a gas chromatograph method for such substances as sulfur compounds, trimethylamine, acetaldehyde, and styrene. On the other hand, a triangle bag test was used for the sensory test.

The triangle bag test is carried out as follows:

Six panels composed of personnel who will actually carry out the test are preselected. They will have passed the screening test to eliminate those with an abnormal sense of smell. The air in the odor bag (capacity of 3 liters) is replaced two or three times with air deodorized by activated carbon and finally, completely filled. The sample gas (gas with offensive odor) is then injected through a syringe to obtain the designated dilution multiple.

Two other bags (odor bags) are also fully filled with the air deodorized by activated carbon and passed to the panels, together with the first bag filled with the gas to be tested. The members of the panel smell these through the nose cone and record the number of the bag which they judge to be giving off an offensive smell. If the answer of the panel is accurate, the dilution multiple (initially approximately three) is increased in a stepwise fashion and the test is closed at the step where the panel's answer becomes inaccurate or impossible. The dilution multiple at that point is recorded.

In this way, the dilution multiple at which the used odorless air brings the offensive odor of the sample to its threshold value (minimum odor unit perceptible by the sense of smell) is taken as the odor unit for that panel. The averaged value of the odor unit obtained from four panels, excluding two panels which show the maximum and minimum values is taken as the odor unit in that particular case.

Calculation method of odor

$$X' = \frac{\log a_1 + \log a_2}{2}$$

where

X' = Threshold value

a_1 = Maximum dilution multiple value at which the panel's answer is accurate

a_2 = Dilution multiple value at which the panel's answer becomes inaccurate or impossible

$Y = 10^X$

Y = Odor unit

$$X = \frac{X_1' + X_2' + X_3' + X_4'}{4}$$

(Averaged value of X' for four panels excluding the maximum and minimum values)

1.1.3 Odor at Sewerage Facilities

Regarding the actual condition of odor unit at sewage treatment plants and pumping stations, one may consult an investigation report1) which indicates that the odor unit is 100 ~ 1,000 in grit chambers when screening, 100 ~ 10,000 in primary sedimentation tanks, 30 ~ 200 at a height 30 cm above the liquid level in aeration tanks, and 200 ~ 1,000 at a height 30 cm above the liquid level in sludge thickening tanks.

In other instances, the occurrence of odor pollution is assumed to be as shown in Table 7 by using O.E.R. (Odor Emission Rate) i.e. the product of emitted gas volume and odor unit.2) According to this table, it can be assumed that in sewerage facilities a small scale odor pollution occur-or will occur-at the value of O.E.R. $10^5 \sim 10^6$.

1) "Investigation Report on Odor and Deodorization" Data No. 1257
Laboratory of Civil Engineering, Ministry of Construction

2) "Basic Problems of Deodorization in the Sewerage Work"
Yoshihiro Shigeta
Journal of Sewerage, Monthly
Vol. 2, No. 8 1979

Table 7 Occurrence Condition of Offensive Odor and OER

Odor Emission Rate (Total OER) [Nm^3/min]	Occurrence Condition of Odor Pollution	Typical Examples of Industry	Remarks
10^4 or below	Will not occur except in special case.	Bakery Brewery	
10^{5-6}	Small scale pollution occurs at present or is a possibility.	Paint manufacturer or painting plant Printing or ink plant Leather plant FRP plant Bait or fertilizer plant Sewage treatment plant	Maximum reaching distance of odor is 1 to 2 km and complaints arise mostly in the range of 500m. It is assumed that complaints will not arise beyond 1 km.
10^{7-8}	Small or medium scale pollution occurs.	Foundry Night soil treatment plant Swinery or poultry Petroleum chemical plant	Maximum reaching distance of odor is 2 to 4 km and complaints for offensive odor arise within 1 km range.
10^{9-10}	Large scale pollution occurs.	KP plant Cellophane plant Slaughterhouse Rayon plant	Maximum reaching distance of odor is within 10 km and complaints for offensive odor arise within the range of 2 to 3 km.
10^{11-12}	Maximum source for occurrence of pollution and actual example is a few.	Large scale KP plant without pollution control	Maximum reaching distance of odor extends to some tens of kilometers, and the range of damage is 4 to 6 km.

Note: This table was compiled from the data produced in the past ten years regarding total OER and pollution through offensive odor by Japan Environmental Sanitation Center.

1.2 History of the Installation of Deodorization Equipment

The first deodorization equipment in Yokohama was installed to control the high concentration odor which was generated by the discharge of night soil from the night soil carrier when it was mixed with raw sludge and treated by digestion. This equipment included an ozone deodorizer and deodorization equipment utilizing the catalytic combustion method.

The deodorization equipment was also required for the high concentration odor generated from the wet air oxidation plant (W.A.O.).

Since then, with the installation of covers for grit chambers and sludge thickening tanks, installation of deodorization equipments as well as ventilation equipments has been mainly proceeded to improve working circumstances.

With the construction of sludge heat dryers and sludge incinerators, it has become necessary to control offensive odor generated from these facilities.

On the other hand, it has become increasingly necessary to consider the immediate environment when constructing pumping stations in dense residential areas where there are tall buildings or where the pumping station is a combined facility with buildings such as public halls and libraries.

In Yokohama, deodorization equipment has been adopted which is appropriate to the working circumstances and immediate environment.

However, continuous and thorough reviews are necessary on odor control since various kinds of odor exist in the sewage treatment plants and pumping stations.

The history of the installation of deodorization equipments in Yokohama is shown in Table 8, and Fig. 1 a ~ m show their flows.

Table 8 History of Installation of Deodorization

Installation Date	Name of Treatment Plant or Pumping Station	Origin of odor	Deodorization Method
May, 1962	Chubu S.T.P.	Night soil discharge room	[Ozone]+[Water]
Dec. 1967	Nambu S.T.P.	Night soil discharge room	[Catalytic combustion]
Feb. 1971	Hokubu 1st S.T.P.	Wet air oxidation plant (Gas-liquid separator and solid-liquid separator	[Catalytic combustion]
Apr. 1971	Hokubu 1st S.T.P.	Wet air oxidation plant, (oxidized slurry pumping room.)	[Carbon]
Apr. 1972	Chubu S.T.P.	Grit chamber	[Ozone]+[Water]
Mar. 1975	Nambu S.T.P.	Sludge heat dryer	[Direct combustion]
Oct. 1976	Midori S.T.P.	Grit chamber	[Water]
Dec. 1977	Hokubu 1st S.T.P.	Sludge thickening tank and others	[Carbon]
Apr. 1978	Kohoku S.T.P.	Sludge storage tank and others	[Water]
June, 1978	Kanazawa S.T.P.	Sludge incinerator	[Direct combustion]
Sep., 1978	Kanagawa S.T.P.	Sludge storage tank and others	[Water]
Oct., 1978	Hodogaya P.S.	Grit chamber	[Carbon]
July, 1979	Totsuka P.S.	Grit chamber	[Water]+ [Ozone]+[Catalyst]
Aug. 1979	Hokubu 1st S.T.P.	Wet air oxidation plant	[Direct combustion] or [Catalytic combustion]

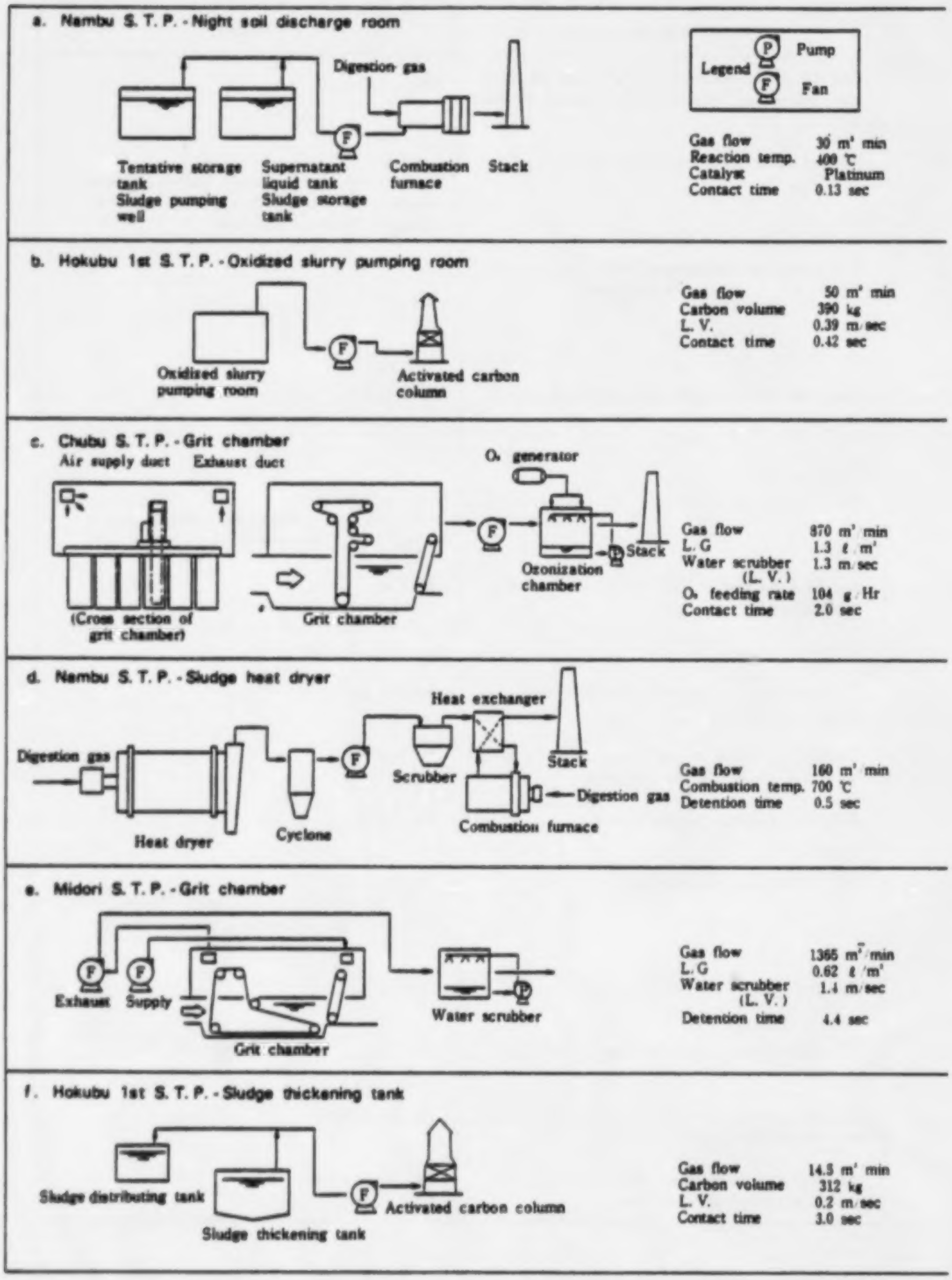
[Water] : Water scrubbing method
[Ozone] : Ozonization method
[Carbon] : Activated carbon method
[Catalyst] : Catalytic ozonization method

Equipment in Yokohama

Reason for Installation	Remarks
Due to high concentration offensive odor generated when discharging night soil from carrier to receiving tank.	
Same as above and also to utilize digestion gas effectively	See Fig. 1, a Basic Law For Environmental Pollution Control was established in August, 1967 and offensive odor was defined as pollution.
Due to high concentration offensive odor generated from WAO equipment. (Exhaust gas)	
Odor unit of oxidized slurry is comparatively low.	See Fig. 1, b
Grit chamber was covered.	See Fig. 1, c Offensive Odor Control Law was established in June 1971 and five substances were regulated.
To utilize digestion gas effectively and due to high odor unit of exhaust gas.	See Fig. 1, d Operational Ordinance of Offensive Odor Control Law and Operational Regulation of the Law were established in May 1972.
To lower odor unit by some degree beside to improve ventilation.	See Fig. 1, e
To make operation and maintenance easier although original odor unit is at medium grade.	See Fig. 1, f
To lower odor unit by some degree.	See Fig. 1, g
Due to high odor unit of exhaust gas	See Fig. 1, h Regulated District and Regulation Standard were established by, Yokohama City Notice No. 3 in May 1978.
To lower odor unit by some degree.	See Fig. 1, i
As residence area is very close although odor unit is low.	See Fig. 1, j
As the facility is in the same building of public hall and the installation area is narrow.	See Fig. 1, k
As catalyst in the installed deodorization furnace lost its activation in a short time.	See Fig. 1, ℓ & m.

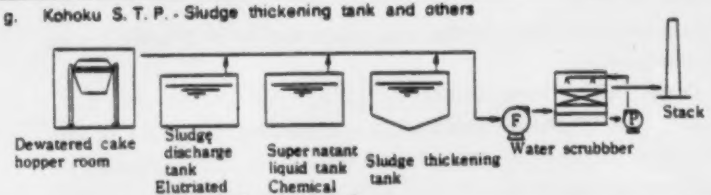
Fig. 1 Flow Sheets of Installed

Fig. 1 Flow Sheets of Installed Deodorization Equipment


a. Nambu S. T. P. - Night soil discharge room
Digestion gas
Tentative storage tank
Sludge pumping well
Supernatant liquid tank
Sludge storage tank
Combustion furnace
Stack
Legend
P Pump
F Fan
Gas flow 30 m³ min
Reaction temp. 400 ℃
Catalyst Platinum
Contact time 0.13 sec
b. Hokubu 1st S. T. P. - Oxidized slurry pumping room
Oxidized slurry pumping room
Activated carbon column
Gas flow 50 m³ min
Carbon volume 390 kg
L. V. 0.39 m/sec
Contact time 0.42 sec
c. Chubu S. T. P. - Grit chamber
Air supply duct
Exhaust duct
(Cross section of grit chamber)
Grit chamber
O₃ generator
Ozonization chamber
Stack
Gas flow 870 m³ min
L. G 1.3 ℓ m³
Water scrubber (L. V.) 1.3 m/sec
O₃ feeding rate 104 g/Hr
Contact time 2.0 sec
d. Nambu S. T. P. - Sludge heat dryer
Digestion gas
Heat dryer
Cyclone
Scrubber
Heat exchanger
Stack
Combustion furnace
Digestion gas
Gas flow 160 m³ min
Combustion temp. 700 ℃
Detention time 0.5 sec
e. Midori S. T. P. - Grit chamber
Exhaust
Supply
Grit chamber
Water scrubber
Gas flow 1365 m³/min
L. G 0.62 ℓ/m³
Water scrubber (L. V.) 1.4 m/sec
Detention time 4.4 sec
f. Hokubu 1st S. T. P. - Sludge thickening tank
Sludge distributing tank
Sludge thickening tank
Activated carbon column
Gas flow 14.5 m³ min
Carbon volume 312 kg
L. V. 0.2 m/sec
Contact time 3.0 sec

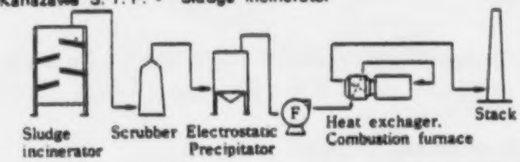
Deodorization Equipment

g. Kohoku S. T. P. - Sludge thickening tank and others


Dewatered cake hopper room
Sludge discharge tank
Elutriated sludge tank
Super natant liquid tank
Chemical mixing tank
Sludge thickening tank
F
Water scrubbber
P
Stack

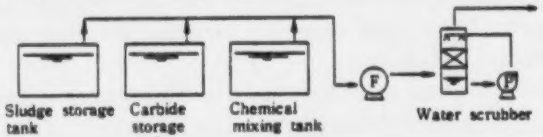
Gas flow	160 m^3/min
L/G	1.88 ℓ/m^3
Water scrubber (L. V.)	0.97 m/sec
Contact time	1.5 sec

h. Kanazawa S. T. P. - Sludge incinerator


Sludge incinerator
Scrubber
Electrostatic Precipitator
F
Heat exchager. Combustion furnace
Stack

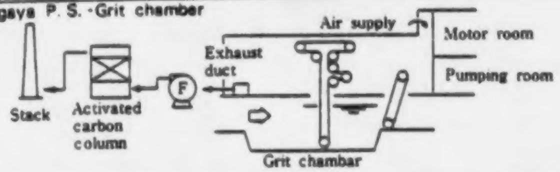
Gas flow	280 m^3/min
Combustion temp.	650~800 ℃
Detention time	0.5 sec

i. Kanagawa S. T. P. - Sludge storage tank and others


Sludge storage tank
Carbide storage tank
Chemical mixing tank
F
P
Water scrubber

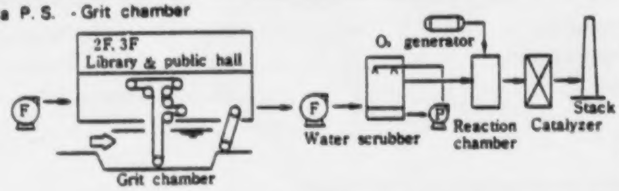
Gas flow	15 m^3/min
L/G	1.65 ℓ/m^3
Water scrubber (L. V.)	1.7 m/sec
Contact time	0.72 sec

j. Hodogaya P. S. - Grit chamber


Air supply
Motor room
Pumping room
Exhaust duct
F
Stack
Activated carbon column
Grit chambar

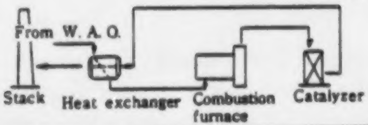
Gas flow	2700 m^3/min
Carbon volume	25000 kg
L. V.	0.435 m/sec
Contact time	1.38 sec

k. Totsuka P. S. - Grit chamber


2F, 3F Library & public hall
F
Grit chamber
F
Water scrubber
O_3 generator
P
Reaction chamber
Catalyzer
Stack

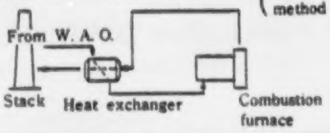
Gas flow	2100 m^3/min
L/G	1.1 ℓ/m^3
L. V.	1.0 m/sec (water) 0.42 m/sec (catalyst)
O_3 feeding rate	600 g/hr
Detention time	3.1 sec (water) 2.6 sec (reaction)
Contact time	0.23 sec. (catalyst)

l. Hokubu 1st S. P. T. - Wet air oxidation (Catalytic combustion method)


From W. A. O.
Stack
Heat exchanger
Combustion furnace
Catalyzer

Gas flow	42 m^3/min
Combustion temp.	400 ℃
Calayst	Platinum
Contact time	0.144 sec

m. Hokubu 1st S. T. P. - Wet air oxidation (Direct combustion method)


From W. A. O.
Stack
Heat exchanger
Combustion furnace

Gas flow	42 m^3/min
Combustion temp.	800 ℃
Detention time	0.8 sec

1.3 Actual Results of Operation

1.3.1 Measured Value of Odor

Table 9 shows the odor values measured during the operation of deodorization equipments in Yokohama.

Table 10 shows the converted value of the each offensive odor substance into the corresponding odor grade index.
The maximum value is shown in bold type letters.

The major component of measured substances was hydrogen sulfide in the sewage treatment system, and methyl mercaptan in the sludge treatment system.

Fig. 2 shows the odor unit based upon the sensory test and the maximum odor grade index. According to this figure, the odor grade index and the odor unit show a high correlation.

Fig. 3 shows the result of deodorization effect indicated by the odor unit. It shows that exclusive treatment by water scrubbing does not give a good effect.

1.3.2 Operation and Maintenance Costs of Deodorization Equipment

The routine operation of deodorization equipment requires little manpower. The exchange work of activated carbon or various catalysts differ with the type of equipment, and although they are of the same type, the expenses incurred will differ according to the scale of the equipment used.

The comparison of direct operation and maintenance costs (power cost, water cost, and articles of consumption) for each type, excluding labor costs and capital costs are shown in Table 11.

In the activated carbon method, the life of activated carbon is assumed to be six months, but its actual life may be longer than that because of the lower odor unit when compared with the designed value. It may be that the operation and maintenance costs fall to the same level as those for the ozonization method.

The operation cost of equipment which use fuel, such as the direct combustion method or the catalystic combusion method, is higher than that for other methods. The operation and maintenance cost for the equipment of catalystic combusion method with the same capacity is 50% less than that for the direct combustion method. When digestion gas is used as fuel i.e. Nambu Sewage Treatment Plant, the cost will largely decrease even with the direct combustion method.

1.4 Problems of Existing Deodorization Equipment

Problems on each treatment methods of existing deodorization equipments are as following.

a. Water scrubbing method

In the case as the Midori Sewage Treatment Plant where the odor unit of original odor is low, offensive odor substances contained in scrubbing water may be stripped to cause the odor unit or the amount of offensive odor substances to be increased.

Even in the case of a sludge storage tank where the odor unit of original gas is considerably high, as in the Kanagawa Sewage Treatment Plant, the removal rate of offensive odor was only around 50%.

Such being the case, the deodorization effect cannot be expected through use

Table 9 Actual Results of Operation of Deodorization Equipments

S.T.P. or P.S.	Origin of Odor	Deodorization method	Items (Measured at)	Odor unit	Substance Concentration (ppm)									Frequency of Sample
					Hydrogen sulfide	Methyl mercaptan	Methyl sulfide	Dimethyl disulfide	Ammonia	Trimethyl amine	Styrene	Acetaldehyde	Formaldehyde	
[illegible] S.T.P.	Grit chamber	[illegible] + (Water)	Inlet Outlet Average (In, out)	16–630 1–490 230, 26	0.005–0.030 0.001–0.027 0.016, 0.0058	0.001–0.007 <0.0001–0.001 0.0034, <0.0007	0.0001–0.003 <0.0001–0.001 0.0017, <0.0001	<0.001–0.001 <0.0001–<0.001 <0.0005, <0.0001	0.022–0.300 0.011–0.10 0.15, 0.047	<0.00005–0.001 <0.00005–0.001 <0.0001, 0.0001				5 4 for trimethyl amine
[illegible] S.T.P.	Grit chamber	(Water)	Inlet Outlet Average	16–110 19–300 93, 97	0.001–0.0192 0.001–0.0792 0.0067, 0.013	<0.0001–0.001 <0.0001–0.0019 <0.0003, <0.0003	<0.0001–0.0107 <0.0001–0.0016 <0.0013, <0.0006	<0.0001–0.0022 <0.0001–0.0013 <0.0004, <0.0004	<0.01–0.6 <0.01–0.2 <0.167, <0.068	<0.00005–0.0003 <0.00005–0.0004 <0.00013, <0.00017				11 5 for ammonia & trimethyl amine
[illegible] P.S.	Grit chamber	(Carbon)	Inlet Outlet Average	21–230 <10–30 103, <17	<0.0001–0.004 <0.0001–0.001 <0.0016, <0.001	0.0001–0.0021 <0.0001–0.0011 0.0007, <0.0003	0.0003–0.0023 0.0001–0.0009 0.001, 0.0003	<0.0001–0.0003 <0.0001–0.0001 <0.0001, <0.0001	0.062–0.6 0.012–0.2 0.23, 0.11	<0.00005–0.0014 <0.00005–0.00006 <0.0004, <0.00006	<0.01 <0.01 <0.01, <0.01	<0.0005 <0.0005 <0.0005, <0.0005		3 2 for styrene & acetaldehyde
[illegible] P.S.	Grit chamber	(Water) + (Ozone) + (Catalyst)	Inlet Outlet Average	126 6	0.02 <0.0001	0.005 <0.0005	0.0023 <0.0005	0.001 <0.0005	0.44 0.36	<0.0005 <0.0005				1
[illegible] S.T.P.	Sludge thickening tank etc.	(Carbon)	Inlet Outlet Average	4000–11000 73–300 13000, 187	0.07–0.36 <0.004–0.032 0.17, <0.007	0.012–0.26 <0.006 0.17, <0.006	0.01–0.07 0.012–0.06 0.035, 0.033	0.0020–0.03 0.011–0.16 0.015, 0.058	0.034–0.15 0.016–0.2 0.12, 0.1	<0.00005–0.0006 <0.00005 <0.0002, <0.00005				5 4 for methyl sulfide, dimethyl disulfide & ammonia
[illegible] S.T.P.	Sludge storage tank etc.	(Water)	Inlet Outlet Average	970 130	0.019 0.0011	0.0044 0.0032	0.0009 0.0007		3.23 0.65	0.0002 <0.0001				1
[illegible] S.T.P.	Sludge storage tank etc.	(Water)	Inlet Outlet Average	3000–13000 3000–13000 26000–7300	0.031–2.2 0.024–0.4 0.81, 0.17	0.012–0.34 0.010–0.16 0.14, 0.109	0.011–0.07 0.01–0.041 0.041, 0.021	<0.01–0.010 <0.01–0.013 0.0077, 0.0057	7.67–10.87 1.76–0.28 14.05, 3.43	0.00022–0.00041 <0.00005–0.00024 0.00032, <0.00014				3
[illegible] S.T.P.	Wet air oxidation plant	Catalytic combustion	Inlet Outlet Average	70000–130000 100–190 290000–130	<0.001 <0.001	<0.001 <0.001	<0.001 <0.001	0.025 <0.0005	<0.2 <0.2	<0.001 <0.001	3.5 <0.04	490 <1	<2 <3	1 4 for odor unit
[illegible] S.T.P.	Wet air oxidation plant	Direct combustion	Inlet Outlet Average	61000–980000 140–230 600000–180								205.6–670 <0.001–0.03 392, <0.016	0.130–0.46 0.04–<0.001 0.283, <0.017	3 4 for odor unit
[illegible] S.T.P.	Sludge heat dryer	Direct combustion	Inlet Outlet Average	13000–20000 410–3000 23000, 1540	Impossible to measure due to SOx effect				70.2–60.7 17.5–42.6 84.7, 27.3	0.044–0.078 0.00014–0.00062 0.055, 0.00026		0.003–1.296 <0.001–0.013 0.489, <0.009	0.034–0.394 0.162–0.267 0.234, 0.211	3
[illegible] S.T.P.	Sludge incinerator	Direct combustion	Inlet Outlet Average	730–23000 380–4300 8300, 1390	<0.01–0.553 <0.001–0.019 1.72, 0.19	<0.005–1.053 <0.005–0.233 <0.066, <0.014	0.316–0.380 0.006–0.036 0.349, 0.021	<0.005–0.033 <0.005–0.007 <0.0275, <0.0045	0.094–95.2 0.577–7.7 47, 2.67	0.00015–0.028 <0.00005–0.00015 0.0088, <0.0001	<0.05 <0.05 <0.05, <0.01	0.003–0.097 <0.001–0.003 0.045, <0.0016	0.074–1.793 <0.001–0.004 0.653, <0.002	5 3 for formaldehyde & acetaldehyde 2 for styrene & methyl sulfide

Table 10 Odor Grade Index of Each Offensive Odor substance

S.T.P. and Sampled Location		Odor Grade Index of Each Offensive Odor Substance							Odor Unit
		Hydrogen Sulfide	Methyl Mercaptan	Methyl Sulfide	Dimethyl Disulfide	Ammonia	Trimethyl Amine	Maximum Odor Grade Index	
Chubu S.T.P. Grit Chamber	Before Deodorization	2.5	2.6	1.4	1.3	<1.0	2.1	2.6	16
		3.7	<1.0	2.2	<1.0	<1.0	<1.0	3.7	300
		2.3	3.3	2.1	1.5	<1.0	<1.0	3.3	170
		2.0	2.2	1.7	1.5	1.9	<1.0	2.2	55
		2.7	3.2	1.7	1.5	<1.0	<1.0	3.2	980
	After Deodorization	1.6	2.0	1.4	1.3	<1.0	2.1	2.1	3
		3.5	<1.0	2.4	<1.0	<1.0	<1.0	3.5	100
		2.2	2.2	1.7	1.5	<1.0	1.7	2.2	980
		1.9	2.2	1.7	1.5	<1.0	<1.0	2.2	170
		2.6	2.2	1.7	1.5	<1.0	<1.0	2.6	550
Midori S.T.P. Grit Chamber	Before Deodorization	1.3	<1.0	<1.0	<1.0	2.0	1.2	2.0	41
		2.8	1.6	1.6	1.3	<1.0	1.4	2.8	80
		2.0	1.8	<1.0	<1.0	-	-	2.0	10
		1.9	<1.0	<1.0	<1.0	-	-	1.9	10
		1.8	1.7	<1.0	<1.0	-	-	1.8	31
		2.4	<1.0	<1.0	<1.0	-	-	2.4	410
		1.6	1.9	1.5	<1.0	<1.0	<1.0	1.9	73
		1.3	2.2	1.7	1.5	<1.0	<1.0	2.2	73
		1.7	2.1	1.7	<1.0	<1.0	<1.0	2.1	130
	After Deodorization	1.6	<1.0	<1.0	<1.0	<1.0	1.5	1.6	130
		3.1	2.6	1.8	1.6	1.2	<1.0	3.1	308
		2.2	2.0	1.3	<1.0	-	-	2.2	36
		2.4	<1.0	1.8	<1.0	-	-	2.4	17
		2.0	<1.0	<1.0	1.3	-	-	2.0	41
		2.5	<1.0	<1.0	<1.0	-	-	2.5	11
		1.3	2.0	1.4	<1.0	<1.0	<1.0	2.0	130
		1.3	2.2	1.7	<1.0	<1.0	1.0	2.2	31
		1.4	1.9	1.7	<1.0	<1.0	1.2	1.9	130
Totsuka P.S. Grit Chamber	Before Deodorization	2.6	3.4	2.0	1.5	2.1	<1.0	3.4	126
	After Deodorization	<1.0	<1.0	<1.0	<1.0	1.7	<1.0	1.7	6
Hodogaya P.S. Grit Chamber	Before Deodorization	1.9	2.2	1.3	<1.0	1.5	2.0	2.2	
		1.8	2.7	2.1	<1.0	2.0	<1.0	2.7	97
		1.2	1.4	1.5	<1.0	<1.0	<1.0	1.5	55
		1.1	1.0	1.5	1.0	1.3	1.2	1.5	98
		<1.0	1.0	1.5	<1.0	<1.0	<1.0	1.5	55
	After Deodorization	1.6	1.6	<1.0	<1.0	<1.0	<1.0	1.6	97
		1.7	2.3	1.7	<1.0	1.2	<1.0	2.3	30
		<1.0	<1.0	1.3	<1.0	1.0	<1.0	1.3	10
		<1.0	<1.0	1.5	<1.0	<1.0	<1.0	1.5	10
		<1.0	<1.0	1.5	<1.0	<1.0	<1.0	1.5	10
Hokubu 1st S.T.P. Sludge Thickening Tank	Before Deodorization	3.1	5.0	2.7	2.0	1.6	<1.0	5.0	4,070
		3.7	5.4	3.2	3.0	<1.0	1.0	5.4	55,000
		3.6	4.9	3.0	2.8	<1.0	<1.0	4.9	9,800
		3.0	5.0	2.5	2.5	<1.0	<1.0	5.0	5,500
	After Deodorization	<1.0	<1.0	2.7	3.2	<1.2	<1.0	3.2	220
		<1.0	<1.0	3.0	3.7	<1.0	<1.0	3.7	170
		<1.0	<1.0	2.6	2.6	<1.0	<1.0	2.6	73
		<1.0	<1.0	3.1	3.3	<1.0	<1.0	3.3	170
[illegible] S.T.P. Sludge Storage Tank	Before Deodorization	2.5	3.0	1.7	-	3.2	1.2	3.2	970
	After Deodorization	1.3	2.8	1.6	-	<1.0	<1.0	2.8	170
Kanagawa S.T.P. Sludge Storage Tank	Before Deodorization	2.7	5.2	3.0	2.8	4.4	1.5	5.2	23,000
		3.5	4.4	2.5	2.2	3.9	1.3	4.4	1,300
		4.5	4.9	3.2	<1.0	4.5	1.4	4.9	41,000
	After Deodorization	2.6	5.0	3.0	2.6	3.9	1.3	5.0	9,800
		3.1	4.3	2.5	2.1	2.8	<1.0	4.3	2,300
		3.8	4.8	2.5	<1.0	3.7	1.1	4.8	5,500
Kanazawa S.T.P. Sludge Incinerator	Before Deodorization	5.0	6.3	3.7	3.0	5.7	2.7	6.3	7,300
		3.0	6.2	3.7	2.9	5.0	2.1	6.2	1,300
		<2.2	<3.4	-	2.8	5.7	3.2	5.7	9,800
		<2.2	<3.1	-	<2.2	<1.0	1.1	-	730
		<2.2	<3.4	-	2.5	3.9	2.6	3.9	23,000
	After Deodorization	2.9	5.2	2.9	1.8	3.9	1.9	5.2	410
		2.6	3.0	2.3	2.4	2.1	1.9	3.0	300
		<2.2	<3.4	-	<2.5	2.0	1.1	-	1,700
		<2.2	<3.1	-	<2.2	3.0	<1.0	-	730
		<2.2	<3.4	-	<2.5	3.0	<1.0	-	5,500

Although the range of odor grade index is 0 to 5, values over 5 as the result of calculation are indicated.
Figures with < mark except <1.0 are due to high limit of detection range.

Fig. 2 Relationship Between Maximum Odor Grade Index and Odor Unit

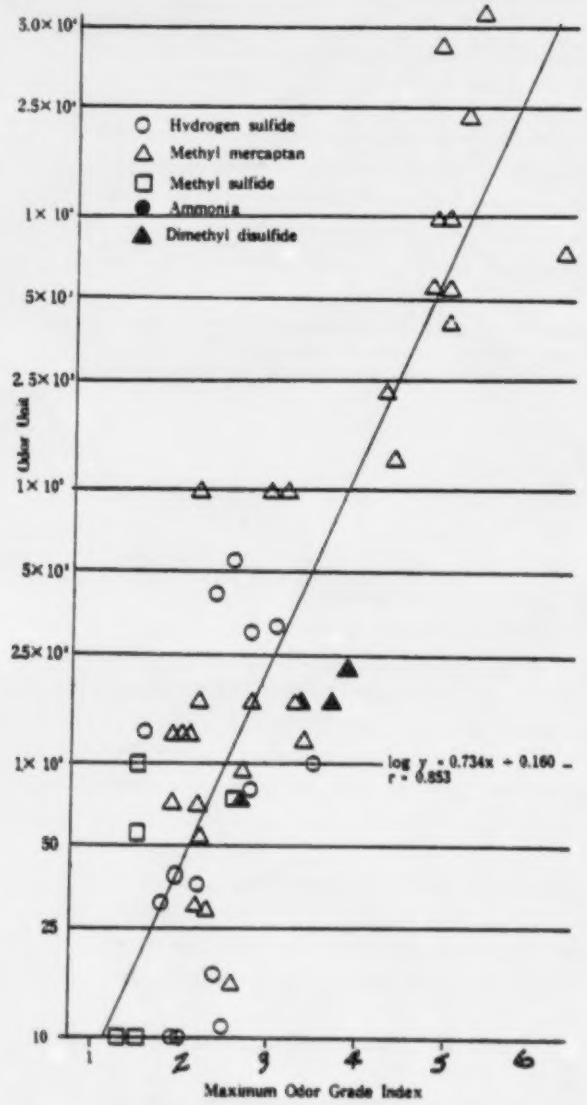


Fig. 3 Removal of Odor Unit of Each Deodorization Equipment

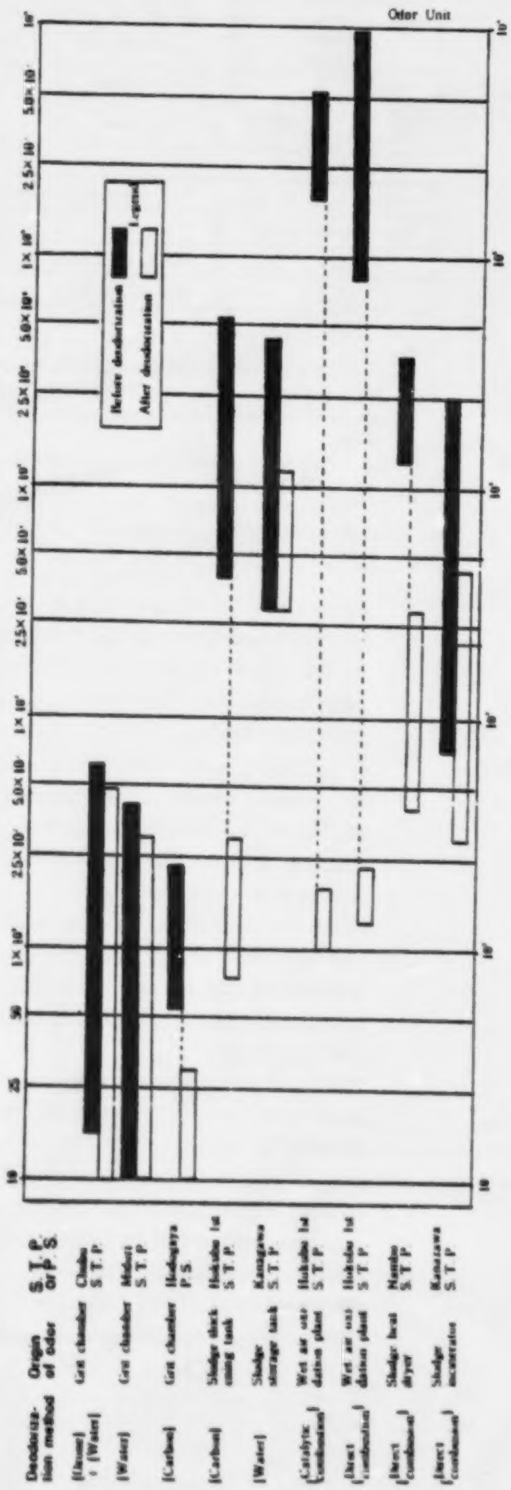


Table 11 Operation and Maintenance Costs in Previously Installed Deodorization Equipment

Deodorization Method	Origin odor	Deodorizing Capacity (m^3/min)	Operation and Maintenance Cost (¥/1,000m^3)
[Water]	Grit chamber	1,365	6.2
[Water] + [Ozone] + [Catalyst]	ditto	2,100	20.9
[Carbon]	ditto	2,700	23.1
[Catalytic combustion]	Wet air oxidation plant	42	993
[Direct combustion]	ditto	42	2,033
ditto	Sludge incinerator	280	1,073
ditto	Sludge heat dryer	160	74.8(1,296)

Notes:

1. Labor cost and capital cost (depreciation, interest) are not included in the operation and maintenance cost.
2. It is assumed that power cost is ¥11/KWH, water cost is ¥200/m^3, and city gas cost is ¥110/m^3.
3. Activated carbon and catalyst are included as articles of consumption and their life times are assumed as designed values.
   Activated carbon .... 6 months
   Platinum catalyst .... 2 years
   Oxidized cobalt .... 2 years
4. In the operation and maintenance cost of Nambu sewage treatment plant, figures shown in parentheses are the converted values into city gas.

of [water] when the odor unit is low. Even in the case of a high odor unit, there is a limitation on the deodorization for this equipment.

In the deodorization by [water], when service water is circulated, offensive odor substances dissolved in water will be stripped of gas and liquid at their saturation points. Water must therefore be exchanged periodically. When secondary treated water is used, it should be used once for all.

By using [water] as a single method, the deodorization effect cannot be expected. It must therefore be considered as a pre-treatment method to be used together with another method.

b. Ozonization method

When the offensive odor in a grit chamber is deodorized by using [ozone] + [water], such major offensive odor substances as hydrogen sulfide and methyl mercaptan can be removed. However, the odor unit after deodorization does not decrease as expected, due to the presence of residual ozone. In cases it even becomes higher.

Ozone threshold value was about 0.005 ppm and, after deodorization, ozone concentration value was about 1.00 ppm.

In order to remove offensive odor substnaces when [ozone] is used, the feeding rate of ozone must be heightened. On the other hand, by making the ozone density higher, activated carbon must be used to remove it, since the odor unit becomes higher due to the residual ozone.

c. Catelytic ozonization method

By using [water] + [ozone] + [catalyst] for deodorization, the effect is difficult to judge because measurement is limited to one occasion and the odor unit of the original gas is as low as 130. However, the odor unit after deodorization is below 10 and the offensive odor substances are removed sufficiently.

In this type of deodorization equipment, it is important to remove mist after water scrubbing to protect the catalyzer and to control the flow to equalize the load on the catalyzer.

d. Activated carbon method

In the deodorization of offensive odor in the grit chamber by using [carbon], although the odor unit of original gas is low, the removal rate reaches over 90% and the offensive odor of sulfide compounds is sufficiently removed. The activated carbon is still not broken after 10 months.

As the result of deodorization of offensive odor from a sludge thickening tank, the removal rate reaches over 90%, although the odor unit varies from 4,000 to 55,000, and the removal rate of hydrogen sulfide and methyl mercaptan also surpasses 90%. There is a tendency for methyl sulfide, dimethyl disulfide, and ammonia increases after deodorization. It is therefor considered that there is a limit to the effect by using activated carbon.

Pre-treatment will be necessary for offensive odor of high concentration and for other substances that cannot be removed off their offensive odor by [carbon].

e. Catalytic combustion method

In the case of the wet air oxidation plant, the original gas of odor unit as high as 520,000 is decreased to 200 or so after deodorization by using the catalytic combustion method.

As the deodorization temperature is lower than [direct combustion], consumption of fuel gas is lower to make the operation and maintenance cost cheaper. However, [catalytic combustion] requires a stopping period of the deodorization equipment to exchange the catalyst, and in facilities generating original gas of high odor unit continuously, spare devices will be required.

f. Direct combustion method

By using this method, the average removal rate of original gas (odor unit close to 10,000) is approximately 80%. Offensive odor substances are almost all removed showing a removal rate of 80 ~ 98%.

However, it is considered that odor unit after the deodorization is not so low as expected because NO_2 increases after deodorization. The threshold value of NO_2 is said to be about 0.03 ppm, and when NO_2 reaches to about 100 ppm, the odor unit will be around 3,000.

The same thing can be said about the deodorization of exhaust gas from the sludge heat dryer. However, the measurement of sulfide compounds is difficult due to the effect of SOx. The concentration of holmaldehyde tends to increase more than that of original gas.

As for the exhaust gas of W.A.O., the odor unit close to 1,000,000 in the original gas decreases to about 200 after deodorization. However the operation and maintenance costs are higher when compared with other equipment.

As the [direct combustion] characteristically generates NOx and it has a notable effect on the odor unit, it is considered that countermeasures against NOx will become important in future.

2. Deodorization Experiments in the Grit Chamber

2.1 Purpose

Offensive odor generated from sewage treatment plants or pumping stations differs in its odor unit and composed substances according to the kind of facility, operation method, and scale. There have been cases where deodorization equipments previously installed did not give good results because of inadequate attention to design or precise onsite requirements.

The city of Yokohama decided to grapple with countermeasure against offensive odor in order to select the best deodorization method. Experiments were first carried out on the grit chambers.

The major reasons were as follows:

a. There are many pumping stations.

b. They have great influence on the surrounding areas as they are located on the urbanized area, and covering the grit chamber, ventilation, deodorization have become necessary.

c. A removal method for offensive odor with a large gas flow and low odor unit such as that found in grit chambers, has not yet been established.

2.2 Plan

2.2.1 Location of Experiment and its Deodorization Method

The three grit chambers at Chubu Sewage Treatment Plant, Isogo Pumping Station, and Totsuka 2nd Sewage Treatment Plant were selected for the experiment.

The criteria for selection were as follows:

a. The grit chamber is covered.

b. The actual flow is close to the design flow.

The following three methods were selected for the experiment.

a. [Chemical] + [Carbon]
   ([Chemical]: Chemical scrubbing method)

b. [Water] + [Ozone] + [Carbon]

c. [Imp. carbon] + [Carbon]
   ([Imp. carbon]: Acid or alkaline impregnated activated carbon)

The reason is described below and various deodorization methods are shown in Fig. 4. The grit chambers selected for this experiment showed a low odor unit because the odor was diluted by using forced ventilation methods.

It is widely known that the activated carbon method is most useful against low odor unit. However, activated carbon is expensive and usually requires pre-treatment to prolong its life by decreasing the constant load on it. Furtheremore, effective deodorization is expected only by applying pre-treatment when low odor unit is generated from the grit chamber, which is subject to a large odor variation with time.

These experiments were carried out by the city in co-operation with three manufacturers of deodorization equipment. Each manufacturer carried out the experiment by installing all three types of equipment at one of the three locations. The purpose of the experiments was to compare each method, and was not to compare the quality of the equipment of each manufacture for a specified deodorization method.

2.2.2 Target Values of Deodorization

The target values of the experiment were determined as 30 or below for the odor unit and 2 or below for the odor grade index at the stack outlet.

The reason why the target value for the odor grade index was determined as 2 is due to the necessity of confirming the deodorization effect to meet the recognition threshold value, because in Yokohama it may be necessary to install the sewerage facilities in densely populated areas.

According to an investigation on odor for the deodorization equipment already installed, the odor unit corresponding to 2 in the odor grade index was about 40 (Fig. 2).

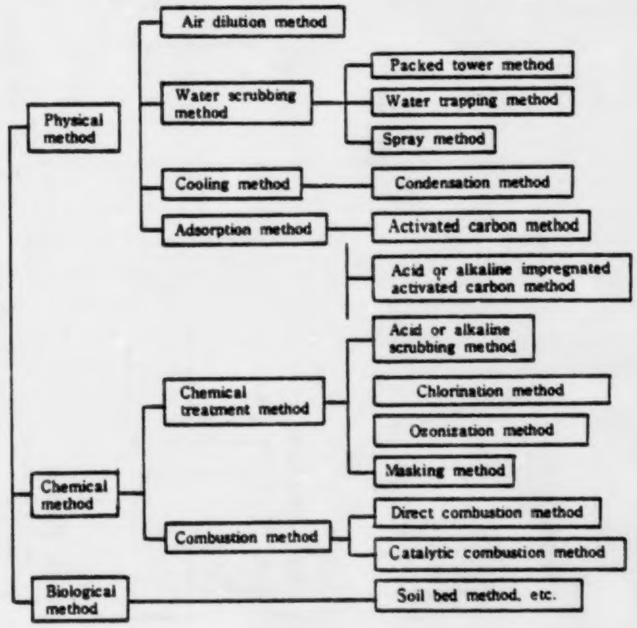
On the other hand, judging from Table 7, "Occurrence Condition of Offensive Odor and O.E.R.", it is considered that odor pollution will not occur if the value is below 1×10^5. The exhaust gas flow in Chubu Sewage Treatment Plant, Isogo Puming Station, and Totsuka 2nd Sewage Treatment Plant were 1600, 4000, 1700 Nm^3/min. respectively. Taking these gas flows into account, the odor unit should be set around 30 in order to make the value of O.E.R. below 1×10^5.

Taking the above into account, the target value of 30 was taken for the odor unit, which is the designated dilution multiple of the triangle bag test.

2.2.3 Outline of Experiment Equipment

Fig. 5 through Fig. 7 show the flow sheets of each deodorization method, Table 12 is explanation of each method and Table 13 through Table 15 show the specifications of major equipment.

Fig. 4 Classification of deodorization methods


Physical method
Air dilution method
Water scrubbing method
Packed tower method
Water trapping method
Spray method
Cooling method
Condensation method
Adsorption method
Activated carbon method
Acid or alkaline impregnated activated carbon method
Chemical method
Chemical treatment method
Acid or alkaline scrubbing method
Chlorination method
Ozonization method
Masking method
Combustion method
Direct combustion method
Catalytic combustion method
Biological method
Soil bed method, etc.

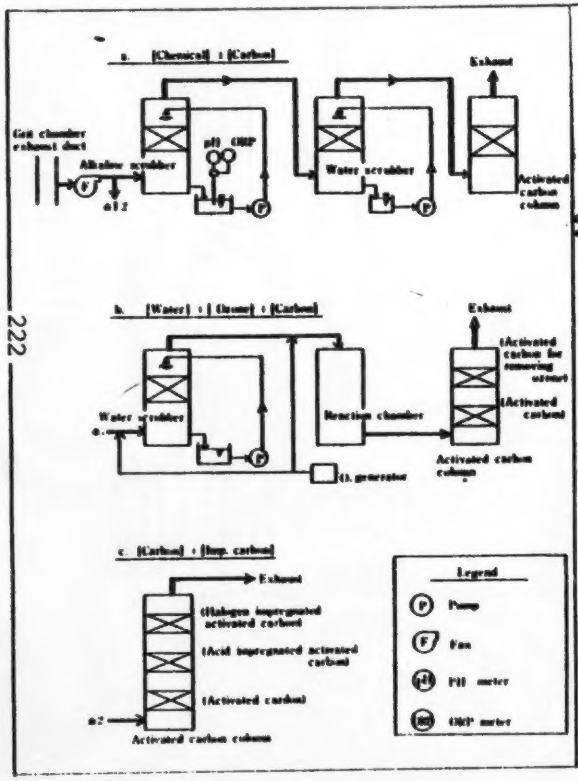

a. [Chemical] + [Carbon]
Grit chamber exhaust duct
Alkaline scrubber
pH ORP
a 1 2
Water scrubber
Exhaust
Activated carbon column
b. [Water] + [Ozone] + [Carbon]
Water scrubber
Reaction chamber
O₃ generator
Exhaust
(Activated carbon for removing ozone)
(Activated carbon)
Activated carbon column
c. [Carbon] + [Imp. carbon]
Exhaust
(Halogen impregnated activated carbon)
(Acid impregnated activated carbon)
(Activated carbon)
a 2
Activated carbon column
Legend
P Pump
F Fan
PH meter
ORP meter

Fig. 5 Flow Sheets of Deodorization Method at Chubu S. T. P.

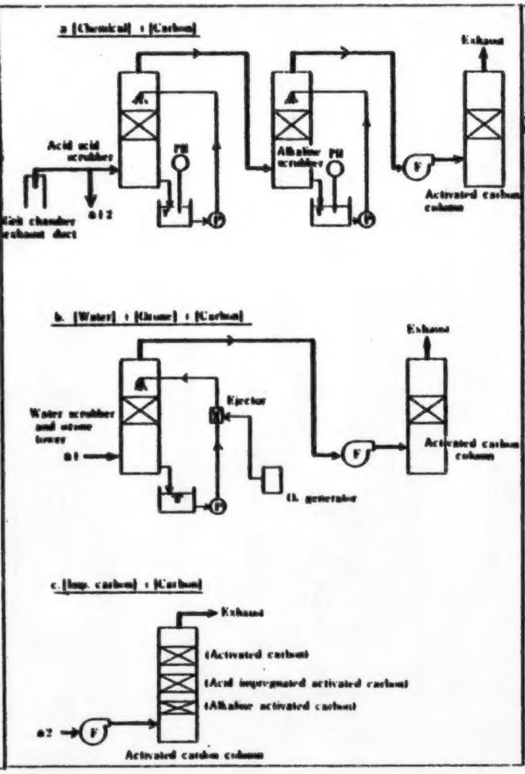

a [Chemical] + [Carbon]
Acid acid scrubber
Grit chamber exhaust duct
a 1 2
PH
Alkaline PH scrubber
Exhaust
Activated carbon column
b. [Water] + [Ozone] + [Carbon]
Water scrubber and ozone tower
a 1
Ejector
O₃ generator
Exhaust
Activated carbon column
c. [Imp. carbon] + [Carbon]
Exhaust
(Activated carbon)
(Acid impregnated activated carbon)
(Alkaline activated carbon)
a 2
Activated carbon column

Fig. 6 Flow Sheets of Deodorization Metho at Isogo P. S.

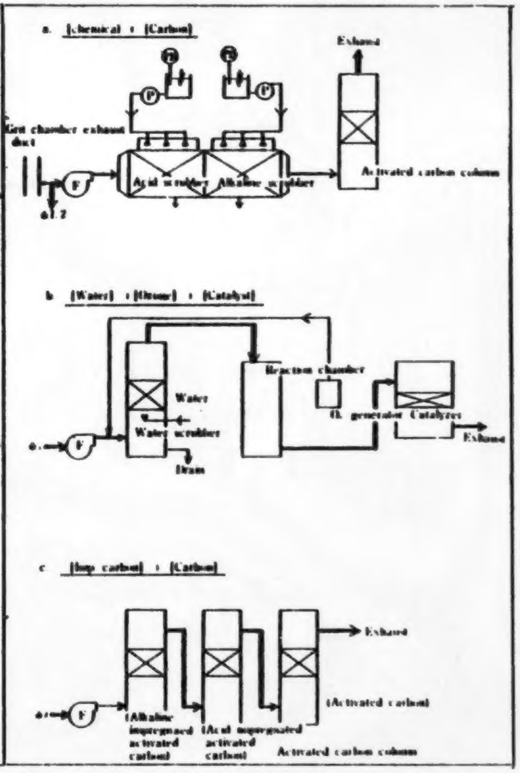

a. [Chemical] + [Carbon]
Grit chamber exhaust duct
a 1 2
Acid scrubber
Alkaline scrubber
Exhaust
Activated carbon column
b [Water] + [Ozone] + [Catalyst]
Water
Water scrubber
Drain
Reaction chamber
O₃ generator
Catalyzer
Exhaust
c [Imp. carbon] + [Carbon]
Exhaust
(Alkaline impregnated activated carbon)
(Acid impregnated activated carbon)
(Activated carbon)
Activated carbon column

Fig. 7 Flow sheets of Deodorization Method at Totsuka 2nd S. T. P.

Table 12 Outline of Deodorization Methods

	Chubu S.T.P.	Isogo P.S.	Totsuka 2nd S.T.P.
[Chemical] + [Carbon]	Chemical scrubbing		
	o Consists of two towers of alkaline scrubbing and water scrubbing. o In alkaline scrubbing, NaOH and NaClO are used to remove acid odor substances and chroline odor is removed by water scrubbing.	o Consists of two towers of acid scrubbing and alkaline scrubbing. o In acid scrubbing H_2SO_4 is used to remove alkaline odor substances. In basic scrubbing the same method as Chubu S.T.P. is used.	o Consists of two towers of acid scrubbing and alkaline scrubbing. o In acid scrubbing, same method as Isogo P.S. is used. Only NaOH is used in alkaline scrubbing.
	Activated carbon		
	o By using pelletized coconut activated carbon, neutral odor substances are adsorbed.	o By using crashed coconut activated carbon, neutral odor substances are adsorbed.	o Same as Chubu S.T.P.
[Water] + [Ozone] + [Carbon]	Water scrubbing + ozonization		
	o Consists of water scrubbing tower and ozone reaction tower. o 1 ppm of ozone is fed before gas enters into each tower of water scrubbing and ozone reaction. o An orifice plate is placed in the ozone reaction tower to mix gas and ozone.	o Consists of only one tower which combines water scrubbing and ozone reaction. o 0.5 ppm of ozone is fed at the time of water scrubbing. o Negative pressure by ejector is used to feed ozone.	o Consists of water scrubbing tower and ozone reaction tower. o 3.9 ppm of ozone is fed into water scrubbing tower. o Volume of water scrubbing is such as to bring humidity to the state where the ozone reaction properly proceeds.
	Activated carbon		Catalyst
	o Pelletized coconut activated carbon is used for adsorbing neutral odor substances and activated carbon to remove the residual ozone are used.	o Crashed coconut activated carbon is used to adsorb neutral odor substances and to remove the residual ozone.	o Catalyst is used to promote the reaction of offensive odor substances with ozone and to remove the residual ozone.
[Imp. carbon] + [Carbon]	First stage		
	o Activated carbon (pelletized coconut) is used to adsorb neutral offensive oder substances.	o Alkaline activated carbon (pelletized coconut) to adsorb acid offensive odor substances is used.	o Alkaline impregnated (pelletized coconut) activated carbon to adsorb acid offensive odor substances is used.
	Second stage		
	o Acid impregnated activated carbon (pelletized coconut) is used to adsorb alkaline offensive odor substances.	o Same as left.	o Same as left.
	Third stage		
	o Halogen impregnated activated carbon (pelletized coconut) is used to adsorb methyl sulfide and dimethyl disulfide.	o Crached activated carbon is used to adsorb neutral offensive odor substances.	o Pelletized activated carbon is used to adsorb neutral offensive odor substances.

Table 13 Specifications of [Chemical] + [Carbon]

Operation Specifications \ Location		Chubu S.T.P.	Isogo P.S.	Totsuka 2nd S.T.P.
Acid scrubbing tower	Gas flow (m^3/min)	4.4	5.0	15.0
	Acid circulation (ℓ/min)	(Water) 2.2	13.0	50.0
	pH	–	3.0	4.0
	Contact time (sec)	0.8	1.4	1.0
	Linear velocity (LV) (m/sec)	1.5	1.18	1.0
	Liquid by gas ratio (L/G)(ℓ/m^3)	0.5	2.6	3.3
	H_2SO_4 (g/Hr)	–	0.66	9.0
Alkaline scrubbing tower	Alkali circulation (ℓ/min)	13.2	13	50.0
	pH	10.0	8.5	11.0
	Contact time (sec)	1.0	1.4	1.0
	Linear velocity (LV) (m/sec)	1.5	1.18	1.0
	Liquid by gas ratio (L/G)(ℓ/m^3)	3.0	2.6	3.3
	NaOH (g/Hr)	6.56	3.38	150.28
	NaClO (g/Hr)	2.78	0.24	–
Activated carbon column	Gas flow (m^3/min)	0.9	5	15
	Linear velocity (LV) (m/sec)	0.3	0.25	0.4
	Contact time (sec)	1.0	1.2	1.4

Table 14 Specifications of [Water] + [Ozone] + [Carbon]

Operation Specifications \ Location		Chubu S.T.P.	Isogo P.S.	Totsuka 2nd S.T.P.
Water scrubbing tower	Gas flow (m^3/min)	3.2	5.0	5.0
	Water (ℓ/min)	1.6	9.7	0 ~ 0.018
	Contact time (sec)	1.1	0.8	1.2
	Linear velocity (LV) (m/sec)	1.1	1.18	0.9
	Liquid by gas ratio (L/G)(ℓ/m^3)	0.5	1.94	0 ~ 0.0036
Ozone reaction tower	Ozone feeding rate (ppm)	2.0	0.5	3.9
	Linear velocity (LV) (m/sec)	1.1	–	0.42
	Contact time (sec)	2.8	–	4.76
Activated carbon column	Gas flow (m^3/min)	0.9	5.0	5.0
	Linear velocity (LV) (m/sec)	0.3	0.25	0.42
	Contact time (sec)	(Carbon) 0.65 (Ozone) 0.5	1.2	0.2

Table 15 Specifications of [Imp. carbon] + [Carbon]

Operation Specifications \ Location		Chubu S.T.P.	Isogo P.S.	Totsuka 2nd S.T.P.
Gas flow (m^3/min)		0.9	5.0	1.67
Linear velocity (LV) (m/sec)		0.3	0.25	0.4
Contact time	First stage	0.67	1.0	1.1
	Second stage	0.67	1.0	1.1
	Third stage	0.67	1.0	1.1

2.2.4 Sampling

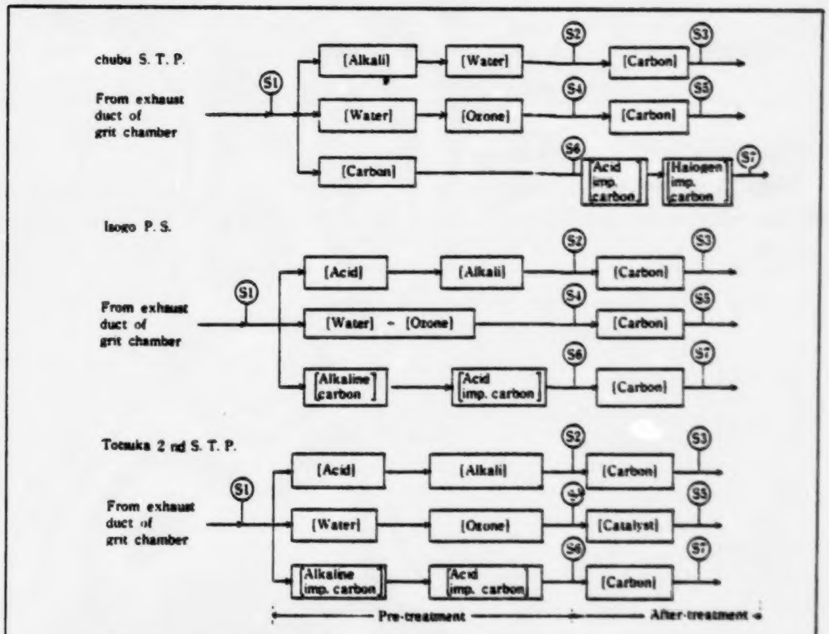
a. Sampling points from where samples were collected were as shown in Fig. 8 in corresponding to deodorization for the grit chamber in operation.

b. Samplings were made while grit dredging was being carried out, and when the offensive odor unit of original gas was assumed to be high. The sampling day was selected as the day following two days of continuing dry weathers.

c. Frequence of sampling was determined as shown below. One of the objects of the experiment was to investigate the breaking down condition of activated carbon. Therefore after three months, the measurement was performed twice a month since the life of activated carbon was assumed to be six months.

Date	1979 Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
Sampling frequency	1	1	1	2	2	2

Fig. 8 Sampling Points


chubu S. T. P.
From exhaust duct of grit chamber
S1
[Alkali]
[Water]
S2
[Carbon]
S3
[Water]
[Ozone]
S4
[Carbon]
S5
[Carbon]
S6
Acid imp. carbon
Halogen imp. carbon
S7
Isogo P. S.
From exhaust duct of grit chamber
S1
[Acid]
[Alkali]
S2
[Carbon]
S3
[Water] - [Ozone]
S4
[Carbon]
S5
Alkaline carbon
Acid imp. carbon
S6
[Carbon]
S7
Totsuka 2 nd S. T. P.
From exhaust duct of grit chamber
S1
[Acid]
[Alkali]
S2
[Carbon]
S3
[Water]
[Ozone]
S4
[Catalyst]
S5
Alkaline imp. carbon
Acid imp. carbon
S6
[Carbon]
S7
Pre-treatment
After-treatment

[Alkali]: Alkaline scrubbing method. [Acid]: Acid scrubbing method.
[Alkaline carbon]: Alkaline activated carbon method.
[Acid imp. carbon]: Acid impregnated activated carbon method.
[Alkaline imp. carbon]: Alkaline impregnated activated carbon method.
[Halogen imp. carbon]: Halogen impregnated activated carbon method

Sampling points of offensive odor are S_1 to S_7.
Sampling point of S_1 to obtain original odor gas.
Sampling points of S_2 S_4 and S_6 to obtain pre-treatment gas.
Sampling points of S_3. S_5 and S_7 to obtain gas at stack outlet.

2.3 Profile of Odor

2.3.1 State of Sewage Treatment at the Experiment Locations

The state of sewage handling at the treatment plants or pumping station which were selected as experiment locations is shown in Table 16. Combined sewerage systems have been adopted at all of these facilities and the major source is domestic sewage.

In the Chubu Sewage Treatment Plant and Totsuka 2nd Sewage Treatment Plant, the effluent from sludge treatment returns into the grit chambers.

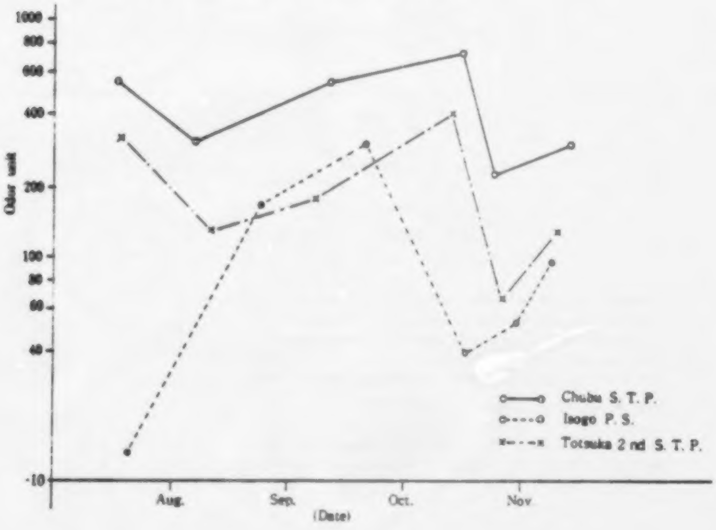
Table 16 State of Sewage Treatment at the Experiment Locations

State of treatment \ Location		Chubu S.T.P.	Isogo P.S.	Totsuka 2nd S.T.P.
Design dry weather flow (m³/day)		64,800	234,000	49,300
Actual dry weather flow (m³/day)		75,900	232,000	37,300
Sludge treatment		Digested sludge dewatering	None	Raw sludge dewatering
Return waste from sludge treatment system		Yes (2.9%)	None	Yes (3.0%)
Ventilation	Method	Forced supply and forced exhaust	Natural supply and forced exhaust	Forced supply and forced exhaust
	Frequency (Times/hour)	3.2 ~ 10.8	12.8	7.6

2.3.2. State of Odor Generation at the Girt Chambers

Table 17 shows the variation range of original odor in the grit chambers. Fig. 9 shows the odor units and Table 18 shows the variation of odor grade index for each substance.

Fig. 9 Odor Unit of Original Gas at Grit Chamber (at Exhaust Duct)



The average odor units at the exhaust duct of the grit chamber of each experimental location were 450, 110, and 240 respectively. As the frequence of ventilation cycles for each experimental location was 3.2, 12.8, and 7.6 times an hour, the values of 1440, 1408, and 1824, which are obtained by multiplying these figures by each odor unit, are assumed as the odor unit at odor source. The odor unit at the conveyor trough of the grit dredger was 1320 (550 – 2310).

The major substances were hydrogen sulfide and methyl mercaptan as sulfur compounds, though each element varies greatly. Methyl sulfide was also generated in rather large quantities.

Methyl mercaptan had been detected at the Chubu Sewage Treatment Plant until September i.e., since the start of the experiment, but it was not detected after October. Trimethyl amine was also detected.

At the Isogo Pumping Station acetaldehyde was also detected.

Table 17 shows the values during the operation of the grit dredger, but Table 19 shows the values of odor for each substance while the grit dredger was out of use at the Totsuka 2nd Sewage Treatment Plant. By operating the grit dredger, it can be seen that the odor concentration varies ten times for hydrogen sulfide and around twice for methyl mercaptan and other substances.

Table 17 State of Odor Generation at the Grit Chamber

		Chubu S.T.P.		Isogo P.S.		Totsuka 2nd S.T.P.	
		Range	Average	Range	Average	Range	Average
Odor unit		230 – 730	450	15 – 310	110	68 – 430	210
Hydrogen sulfide	Concentration (ppm)	0.008 – 0.44	0.11	0.0004 – 0.3	0.073	0.038 – 0.41	0.18
	Odor grade index	2.2 – 3.8	2.8	1.0 – 3.6	2.5	2.8 – 3.8	3.3
Methyl mercaptan	Concentration (ppm)	N.D. – 0.011	0.0033	N.D. – 0.0075	0.0028	0.0004 – 0.023	0.0058
	Odor grade index	<1 – 3.5	2.0	<1 – 3.3	2.4	1.7 – 3.9	2.8
Methyl sulfide	Concentration (ppm)	N.D. – 0.0036	0.0012	N.D. – 0.004	0.0021	0.0012 – 0.0096	0.0034
	Odor grade index	<1 – 2.1	1.6	<1 – 2.2	1.8	1.8 – 2.5	2.0
Dimethyl disulfide	Concentration (ppm)	N.D. – 0.0019	0.0008	N.D. – 0.0016	0.0005	0.0002 – 0.0023	0.001
	Odor grade index	<1 – 1.8	1.3	<1 – 1.7	1.2	<1 – 1.9	1.4
Ammonia	Concentration (ppm)	0.11 – 0.3	0.19	0.1 – 1.9	0.75	0.05 – 0.14	0.095
	Odor grade index	<1 – 1.5	1.2	<1 – 2.9	2.0	<1 – 1.0	<1
Trimethyl amine	Concentration (ppm)	0.001 – 0.11	0.025	N.D. – 0.0001	<0.0001	N.D. – 0.0004	0.00015
	Odor grade index	1.9 – 3.7	2.6	<1 – 1.0	<1	<1 – 1.5	1.2
Acetaldehyde	Concentration (ppm)	N.D.		N.D. – 0.2	0.064	N.D.	
	Odor grade index	<2		<2 – 3.3	2.5	<2	
Styrene	Concentration (ppm)	N.D.		N.D. – 0.031	0.011	N.D.	
	Odor grade index	<1		<1 – 1.0	<1	<1	

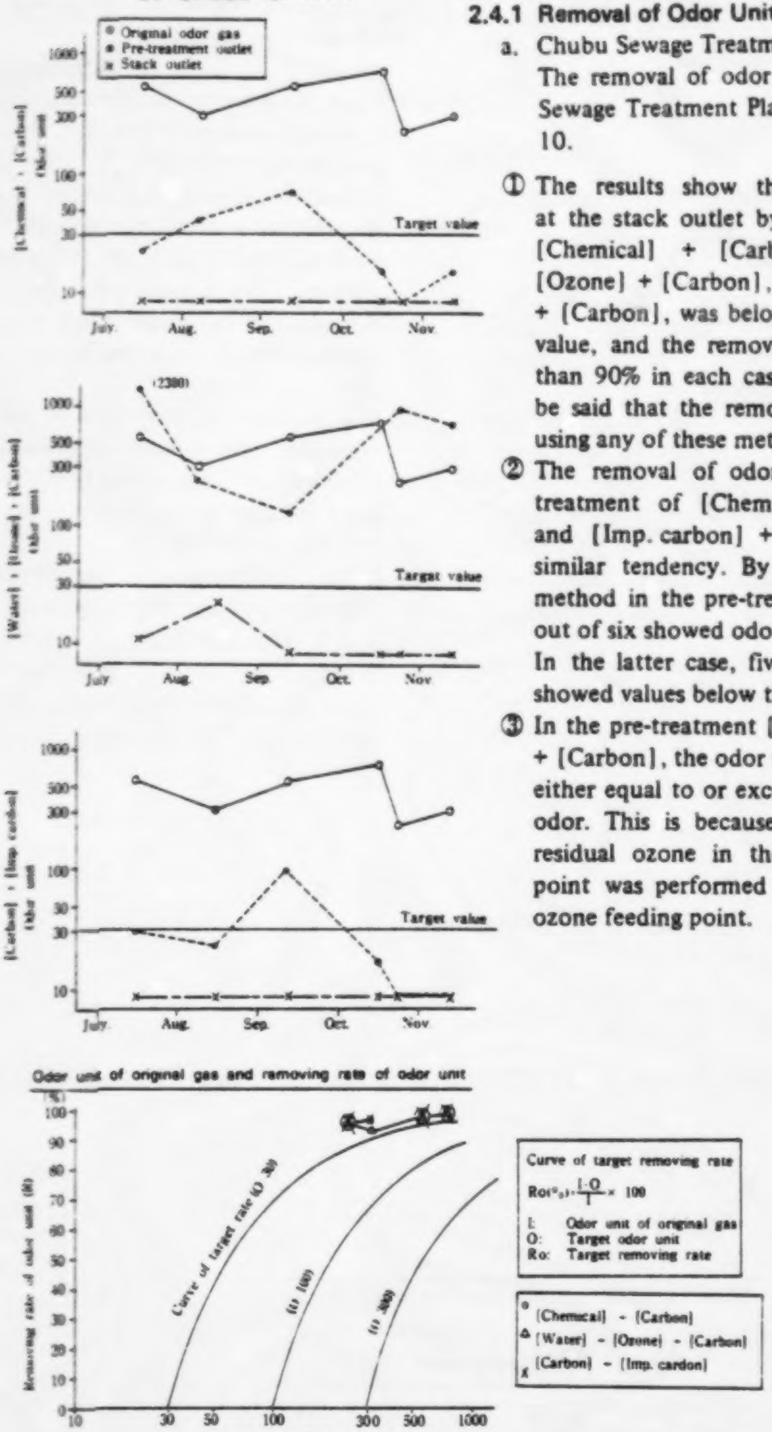
Table 18 Odor Grade Index of Each Substance at the Grit Chamber

Location / Date / Substance	Chubu S.T.P.						Isogo P.S.						Totsuka 2nd S.T.P.					
	July 16	Aug. 6	Sep. 11	Oct. 15	Oct. 23	Nov. 13	July 19	Aug. 23	Sep. 20	Oct. 16	Oct. 29	Nov. 8	July 17	Aug. 10	Sep. 7	Oct. 12	Oct. 25	Nov. 9
Hydrogen sulfide	2.3	3.0	3.1	2.5	2.2	2.7	1.0	2.8	3.1	2.6	2.2	3.0	2.3	3.2	3.0	2.8	2.8	2.7
Methyl mercaptan	2.9	3.4	3.8	<1	<1	<1	<1	2.9	3.3	3.0	1.6	2.5	2.9	2.3	1.7	2.9	2.7	3.1
Methyl sulfide	1.3	2.1	1.6	1.9	1.6	<1	<1	2.2	2.3	1.7	2.1	1.7	1.8	1.8	2.0	2.5	2.2	1.8
Dimethyl disulfide	1.0	1.3	1.7	1.8	1.0	<1	<1	1.5	1.7	<1	<1	1.0	<1	1.1	1.0	1.4	1.9	1.8
Ammonia	<1	1.2	<1	1.5	<1	1.4	2.9	2.4	1.5	<1	2.0	2.0	<1	<1	1.0	<1	<1	<1
Trimethyl amine	2.5	2.3	2.7	2.9	1.9	1.9	1.0	<1	<1	<1	<1	<1	1.5	1.2	<1	<1	<1	1.4
Acetaldehyde	<2	-	-	<2	-	-	3.3	2.9	<2	2.9	<2	2.7	<2	<2	<2	<2	<2	<2
Styrene	<1	-	-	-	-	-	<1	<1	1.0	<1	<1	<1	<1	<1	<1	<1	<1	<1

Table 19 State of Odor generation at Totsuka 2nd S.T.P. when Grit Dredger is not operated

	Substance concentration (ppm)		Odor grade index	
	Range	Average	Range	Average
Hydrogen sulfide	0.0014 ~ 0.038	0.015	1.4 ~ 2.8	2.1
Methyl marcaptan	0.0021 ~ 0.0037	0.0027	2.6 ~ 2.9	2.7
Methyl sulfide	0.0006 ~ 0.0043	0.002	1.5 ~ 2.2	1.8
Dimethyl disulfide	N.D. ~ 0.0023	0.00077	<1 ~ 1.9	1.3
Ammonia	0.01 ~ 0.09	0.04	<1	<1

Fig. 10 Removal of Odor Unit at Chubu S. T. P.



2.4 Effects of Deodorization

2.4.1 Removal of Odor Unit

a. Chubu Sewage Treatment Plant

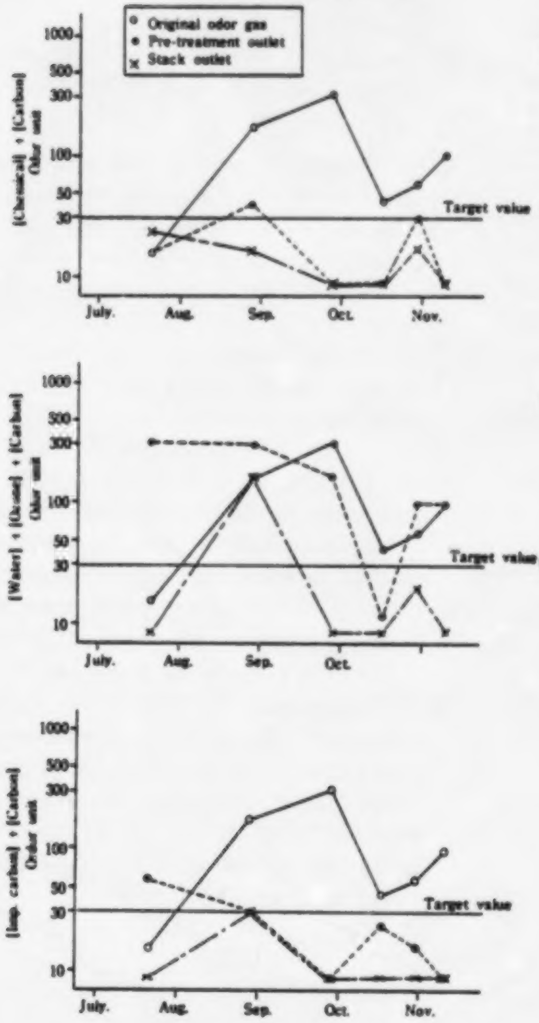
The removal of odor unit at the Chubu Sewage Treatment Plant is shown in Fig. 10.

① The results show that the odor unit at the stack outlet by each of method, [Chemical] + [Carbon], [Water] + [Ozone] + [Carbon], or [Imp. carbon] + [Carbon], was below 30 of the target value, and the removing rate was more than 90% in each case. It can therefore be said that the removal is stable when using any of these methods.

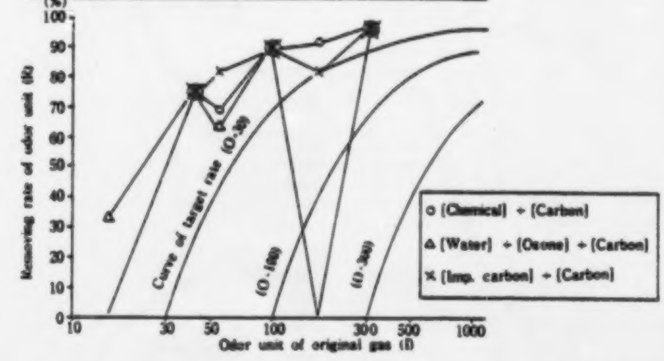
② The removal of odor unit in the pre-treatment of [Chemical] + [Carbon] and [Imp. carbon] + [Carbon] showed similar tendency. By using the former method in the pre-treatment, four cases out of six showed odor unit of below 30. In the latter case, five cases out of six showed values below the target values.

③ In the pre-treatment [Water] + [Ozone] + [Carbon], the odor unit showed values either equal to or exceeding the original odor. This is because of the effect of residual ozone in the gas as sampling point was performed immediately after ozone feeding point.

Fig. 11 Removal of Odor Unit at Isogo P. S.



Odor unit of original gas and removing rate of odor unit



b. Isogo Pumping Station

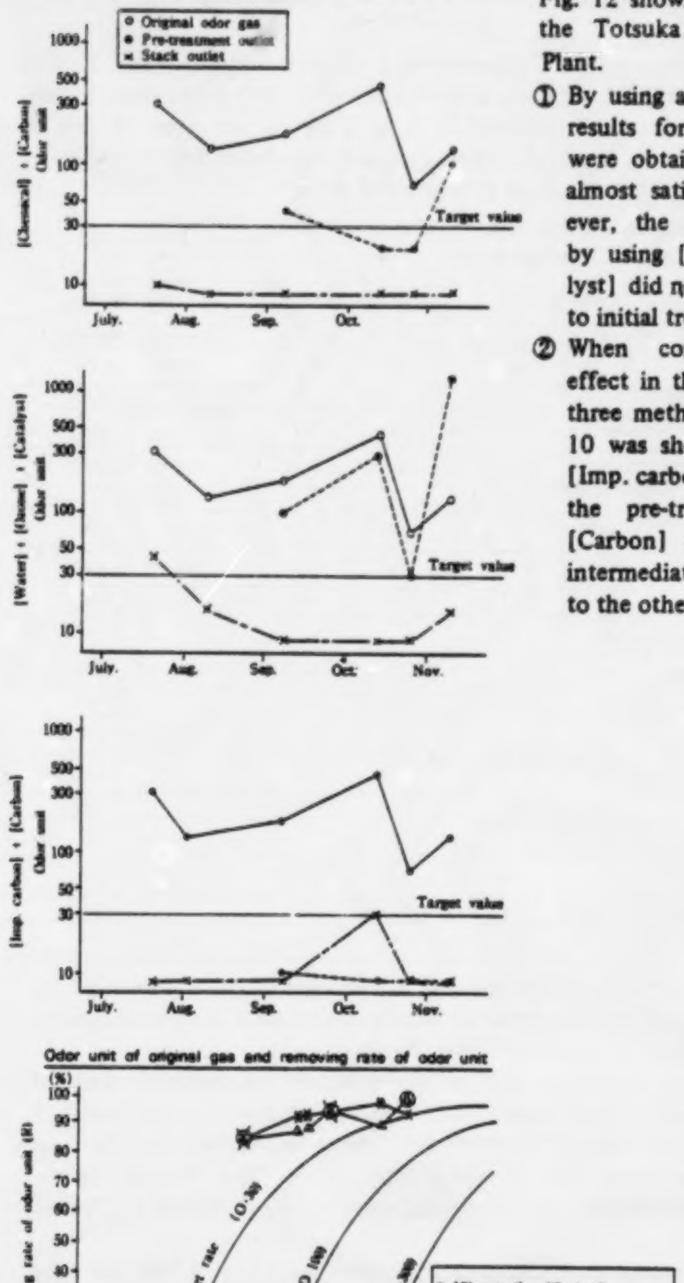
Fig. 11 shows the removal of odor unit at the Isogo Pumping Station.

① By using the three methods, the target value of odor unit was almost attained at the stack outlets. Although the second measurement (August 31) by using [Water] + [Ozone] + [Carbon] showed that the value exceeded the target at the stack outlet, the measured values on and after the third measurement (September 20) satisfied the target value, since the feeding rate of ozone was limited to 0.5 ppm.

② The removal of odor unit in the pre-treatment by using [Chemical] + [Carbon] or [Imp. carbon] + [Carbon] showed a similar tendency and the odor unit was around 30.

③ In the pre-treatment using [Water] + [Ozone] + [Carbon], the odor unit gave a value similar to, or exceeding the original odor value. This is due to the effect of residual ozone and is similar to the case of Chubu Sewage Treatment Plant.

Fig. 12 Removal of Odor Unit at Totsuka 2nd S. T. P.



c. Totsuka 2nd Sewage Treatment Plant

Fig. 12 shows the removal of odor unit at the Totsuka 2nd Sewage Treatment Plant.

① By using any of three methods, similar results for the removal of odor unit were obtained at the stack outlet and almost satisfied the target value. However, the first measurement (July 7) by using [Water] + [Ozone] + [Catalyst] did not satisfy the target value, due to initial troubles.

② When comparing the deodorization effect in the pre-treatment by using the three methods, an odor unit of around 10 was shown in the pre-treatment of [Imp. carbon] + [Carbon]. The result in the pre-treatment of [Chemical] + [Carbon] showed the tendency to be intermediately effective when compared to the other two methods.

2.4.2 Removal of Offensive Odor Substances

a. Chubu Sewage Treatment Plant

Fig. 13 shows the removal of offensive odor substances at the Chubu Sewage Treatment Plant.

Table 20 shows the removal of offensive odor substances and the value of odor grade index. Hydrogen sulfide was almost removed in the pre-treatment in case [Chemical] + [Carbon] or [Carbon] + [Imp. Carbon] was used. In case of [Water] + [Ozone] † [Carbon], after treatment was necessary to reach the total removing rate of 99% obtained by other method.

Table 20 Removal of Offensive Odor Substances and Odor Grade Index at Chubu S.T.P. (Average value)

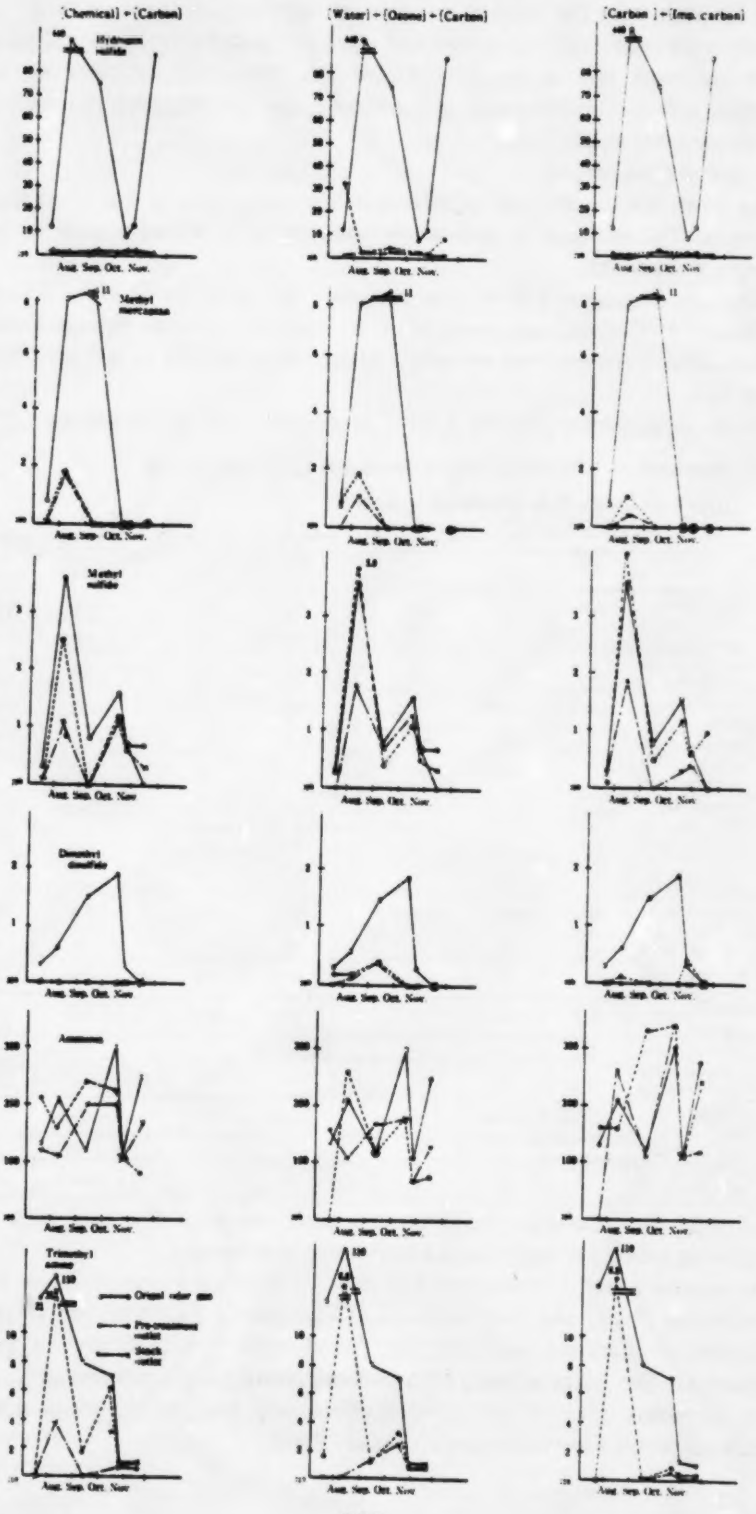
		Original odor	[Chemical]+[Carbon]		Total	[Water]+[Ozone]+[Carbon]		Total	[Carbon]+[Imp. carbon]		Total
			Pre-treatment	After-treatment		Pre-treatment	After-treatment		Pre-treatment	After-treatment	
Hydrogen sulfide	Concentration (ppm)	0.11	0.0014	0.0011		0.0089	0.0019		0.0013	0.0011	
	Odor grade index	2.8	1.4	1.3		2.0	1.5		1.4	1.3	
	Removing rate of concentration (%)		98.7	21.4	99	91.9	78.7	98.3	98.8	15.4	99
Methyl mercaptan	Concentration (ppm)	0.0033	0.0003	0.0003		0.00047	0.00018		0.00018	0.00007	
	Odor grade index	2.0	1.3	1.3		1.5	1.1		1.2	<1	
	Removing rate of concentration (%)		90.9	–	90.9	85.8	61.7	94.5	94.5	62.8	98
Methyl sulfide	Concentration (ppm)	0.0012	0.00087	0.00053		0.0015	0.00077		0.0013	0.00049	
	Odor grade index	1.6	1.5	1.4		1.7	1.5		1.6	1.3	
	Removing rate of concentration (%)		27.5	39.1	33.8	–	48.7	35.8	–	62.3	59.2
Dimethyl disulfide	Concentration (ppm)	0.0008	N.D.	N.D.		0.00013	0.00009		0.00009	N.D.	
	Odor grade index	1.3	<1	<1		<1	<1		<1	<1	
	Removing rate of concentration(%)		100		100	83.8	29.2	88.5	89.2	100	100
Ammonia	Concentration (ppm)	0.19	0.17	0.15		0.12	0.14		0.20	0.18	
	Odor grade index	1.2	1.2	1.1		1.0	1.0		1.2	1.2	
	Removing rate of concentration(%)		10.5	11.8	21.1	36.8	–	26.4	–	10	5.3
Trimethyl amine	Concentration (ppm)	0.025	0.011	0.0015		0.013	0.00097		0.011	0.00018	
	Odor grade index	2.6	2.2	1.5		2.2	1.7		1.7	1.2	
	Removing rate of concentration (%)		56	86.4	94	48	92.5	96.1	56	98.4	99.3

It was possible to remove methyl mercaptan to a level of more than 90%. By using [Chemical] + [Carbon], about 90% can be removed in the pre-treatment, but it is not so effectively removed in the after-treatment.

As for the removal of methyl sulfide, the effect of pre-treatment was not enough by using [Water] + [Ozone] + [Carbon] or [Carbon] + [Imp. carbon], and the removing rate using [Chemical] + [Carbon] was below 30%. The total removing rate was about 50% by using [Carbon] + [Imp. carbon] or the [Chemical] + [Carbon] and was below that value by using [Water] + [Ozone] + [Carbon].

In the case of dimethyl disulfide, since its original odor unit is low, the odor was removed 100% by using either [Chemical] + [Carbon] or the [Carbon] + [Imp. carbon]. The effect of [Water] + [Ozone] + [Carbon] was inferior to these methods. However, by using [Chemical] + [Carbon], the odor was removed 100% only in the pre-treatment.

Fig. 13 Removal of Substances at Chubu S. T. P.



As for ammonia, the removal was low by using any of these methods. This is because its original odor is so low and close to the detective threshold value.

The removing rate of trimethyl amine was above 90% by using any of the methods, but after-treatment was necessary as the removing rate in the pre-treatment was below 60%.

b. Isogo Pumping Station

Fig. 14 shows the removal of offensive odor substances at the Isogo Pumping Station. The removal of substances and the value of odor grade index are shown in Table 21.

Although hydrogen sulfide was removed by some 98%, when [Water] + [Ozone] + [Carbon] was used, as in the case of the Chubu Sewage Treatment Plant, after-treatment was necessary as the removing rate in the pre-treatment was 70%.

The total removing rate of methyl mercaptan was as high as 98 to 100%.

Table 21 Removal of Offensive Odor Substances and Odor Grade Index at Isogo P.S. (Average Value)

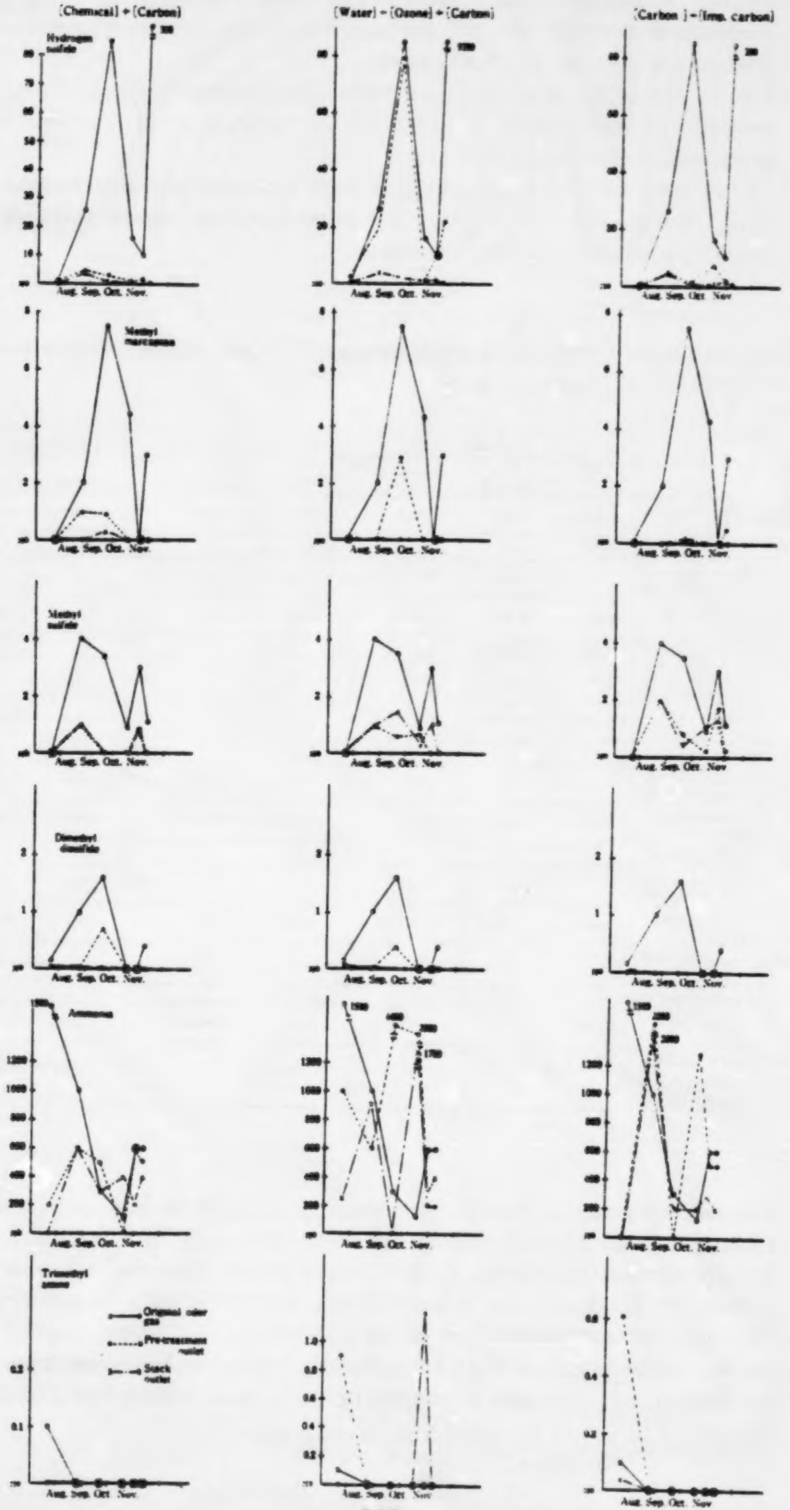
		Original odor	[Chemical]+[Carbon]		Total	[Water]+[Ozone] +[Carbon]		Total	[Imp. carbon]+[Carbon]		Total
			Pre-treatment	After-treatment		Pre-treatment	After-treatment		Pre-treatment	After-treatment	
Hydrogen sulfide	Concentration (ppm)	0.073	0.002	0.0014		0.022	0.0016		0.0029	0.0016	
	Odor grade index	2.5	1.5	1.5		[illegible]	1.4		1.6	1.3	
	Removing rate of concentration (%)		97.3	30	98.1	69.9	92.7	97.8	96	44.8	97.8
Methyl mercaptan	Concentration (ppm)	0.0028	0.00032	0.00005		0.00048	N.D.		0.0001	0.00002	
	Odor grade index	[illegible]	1.4	<1		1.3	<1		1.2	<1	
	Removing rate of concentration (%)		88.6	84.4	98.2	82.9	100	100	96.4	83	99.4
Methyl sulfide	Concentration (ppm)	0.0021	0.00032	0.00032		0.00038	0.00065		0.00082	0.0008	
	Odor grade index	1.8	1.2	1.2		1.4	1.4		1.5	1.6	
	Removing rate of concentration (%)		84.8	0	84.8	81.9	-	69	61	0	61.9
Dimethyl disulfide	Concentration (ppm)	0.0005	0.00013	0.00001		0.00008	0.00001		N.D.	N.D.	
	Odor grade index	1.2	<1	<1		<1	<1		<1	<1	
	Removing rate of concentration (%)		74	92.3	98	84.4	84.6	97.6	100	0	100
Ammonia	Concentration (ppm)	0.75	0.4	0.3		1.4	0.59		0.85	0.58	
	Odor grade index	2.0	1.7	1.5		[illegible]	1.8		1.9	1.5	
	Removing rate of concentration (%)		46.7	25	60	-	57.9	16	-	31.8	22.7
Trimethyl amine	Concentration (ppm)	<0.0001	<0.0001	<0.0001		0.0001	N.D.		0.00015	0.0002	
	Odor grade index	<1	<1	<1		<1	<1		1.1	1.2	
	Removing rate of concentration (%)		0	0	0	0	100	100	0	0	0

Although the removing rate in the pre-treatment was 80%, over 80% of the remaining substance was removed in the after-treatment.

The removing rate of methyl sulfide differed from the result of Chubu Sewage Treatment Plant, and was 80% of [Chemical] + [Carbon] or [Water] + [Ozone] + [Carbon] and 60% by [Imp. carbon] + [Carbon] in the pre-treatment. The effect in the after-treatment, however, was not enough.

The removing rate of dimethyl disulfide was high and showed a similar tendency at the Chubu Sewage Treatment Plant.

Fig. 14 Removal of Substances at Isogo P. S.



Definite results for ammonia were not obtained (Fig. 14) because the variation of measured values of both original gases and treatment gases was so wide.

c. Totsuka 2nd Sewage Treatment Plant

Fig. 15 shows the removal of offensive odor substances at the Totsuka 2nd Sewage Treatment Plant. The removal of substances and the value of odor grade index are shown in Table 22.

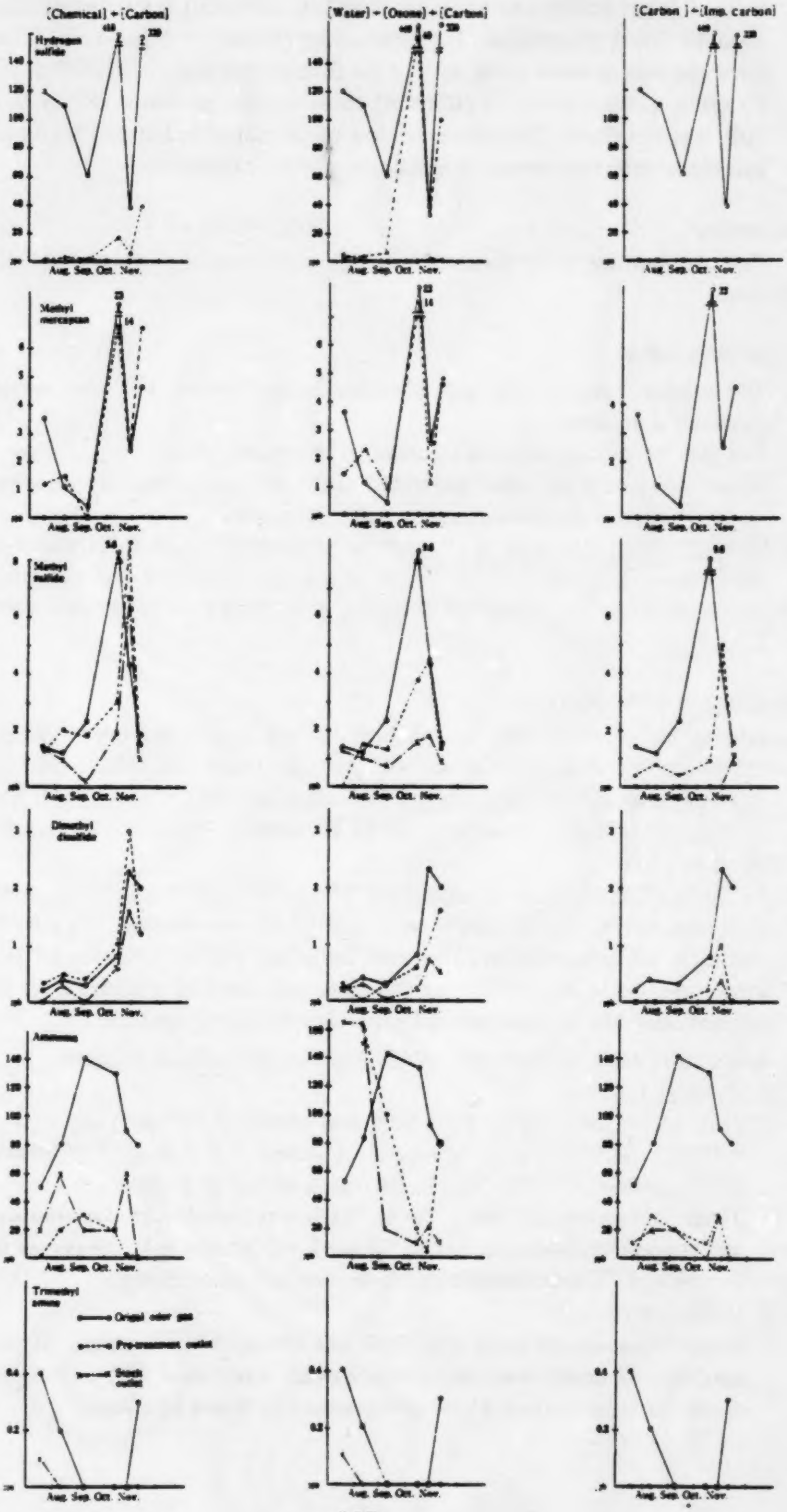
The removal of hydrogen sulfide showed a similar tendency to that at the Chubu Sewage Treatment Plant, and Isogo Pumping Station. Particularly the removal in the after-treatment was high.

Table 22 Removal of Offensive Odor Substances and Odor Grade Index at Totsuka 2nd S.T.P. (Average Value)

		Original odor	[Chemical]+[Carbon]		Total	[Water]+[Ozone] +[Carbon]		Total	[Imp. carbon]+[Carbon]		Total
			Pre-treatment	After-treatment		Pre-treatment	After-treatment		Pre-treatment	After-treatment	
Hydrogen sulfide	Concentration (ppm)	0.18	0.011	0.00028		0.064	0.0013		0.00007	N.D.	
	Odor grade index	3.5	1.8	1.2		2.5	1.3		<1	<1	
	Removing rate of concentration (%)		93.9	97.5	99.8	64.4	98	99.3	100	0	100
Methyl mercaptan	Concentration (ppm)	0.0058	0.0043	N.D.		0.0041	N.D.		N.D.	N.D.	
	Odor grade index	2.8	2.6	<1		2.7	<1		<1	<1	
	Removing rate of concentration (%)		25.9	100	100	29.3	100	100	100	0	100
Methyl sulfide	Concentration (ppm)	0.0034	0.0031	0.002		0.0021	0.0012		0.0014	0.00017	
	Odor grade index	2.0	2.0	1.8		1.8	1.8		1.7	1.1	
	Removing rate of concentration (%)		8.8	35.5	41.2	38.2	42.9	64.7	58.8	87.9	95
Dimethyl disulfide	Concentration (ppm)	0.001	0.0011	0.00062		0.00063	0.00032		0.0002	0.00007	
	Odor grade index	1.4	1.4	1.3		1.3	1.1		1.1	<1	
	Removing rate of concentration (%)		-	43.6	38	37	49.2	68	80	66.5	93.9
Ammonia	Concentration (ppm)	0.095	0.012	0.03		0.061	0.082		0.015	0.013	
	Odor grade index	<1	<1	<1		<1	<1		<1	<1	
	Removing rate of concentration (%)		87.4	-	68.4	35.8	-	13.7	84.2	13.3	86.3
Trimethyl amine	Concentration (ppm)	0.00015		0.000017			0.000017			N.D.	
	Odor grade index	1.2		<1			<1			<1	
	Removing rate of concentration (%)				88.9			88.9			100

The removing rate of methyl mercaptan by using [Chemical] + [Carbon] or [Water] + [Ozone] + [Catalyst] was below 30% in the pre-treatment, but in the after-treatment showed a 100% removal rate. However, by using [Imp. carbon] + [Carbon], the removing rate was 100% only in pre-treatment. The effect of pre-treatment by using [Chemical] + [Carbon] seemed rather inferior to the results at the Chubu Sewage Treatment Plant, and Isogo Pumping Station, but the cause is assumed to be because sodium hypochlorite was not used in this process.

Fig. 15 Removal of Substances at Totsuka 2nd S. T. P.



As for methyl sulfide and dimethyl disulfide, the total removing rate was not so good by using [Chemical] + [Carbon] or [Water] + [Ozone] + [Catalyst], since the results were about 40% in the former case and about 60% in the latter. By using [Imp. carbon] + [Carbon] however, the removing rate of more than 90% was obtained. The concentration of ammonia and trimethyl amine were below the detective threshold values even in the original odor.

2.5 Summary

The observations from the deodorization experiments at the grit chambers are as follows:

2.5.1 Original Odor

a. The original odor at the grit chamber largely varies with the operation of mechanical equipments.

b. The generated odor unit was assumed to be around 1500.

c. Major substances of odor generated from the grit chamber were hydrogen sulfide and methyl mercaptan as sulfur compounds.

d. In some cases, the odor grade indices of methyl sulfide and trimethyl amine were above 2 at the exhaust ducts of the grit chambers but the odor grade indices of ammonia, dimethyl disulfide, acetaldehyde, and styrene were below 2.

2.5.2 Effect of Deodorization

a. The values of odor unit were below 30, which was the target value of the experiment, and sometimes showed a value below 10, which was the limit value of measurement by using any particular method ([Chemical] + [Carbon], [Water] + [Ozone] + [Carbon], or [Imp. carbon] + [Carbon]). Results seemed to be good.

b. By using [Chemical] + [Carbon] or [Imp. carbon] + [Carbon], there were some cases in which the target odor unit of 30 was obtained only by the pre-treatment of deodorization. However, by using [Water] + [Ozone] + [Carbon], some cases were observed when the odor unit reached a higher level than the original odor due to the residual ozone after the pre-treatment.

c. Removal of each offensive odor substance was generally as follows:

① Hydrogen sulfide

A removing rate higher than 98% was obtained by using any of the three methods ([Chemical] + [Carbon], [Water] + [Ozone] + [Carbon], and [Imp. carbon] + [Carbon]). By using either [Chemical] + [Carbon] or [Imp. carbon] + [Carbon], 94 to 100% was removed in the pre-treatment, but the removal was inferior to [Water] + [Ozone] + [Carbon] as the rate was 65 – 92%, and therefore after-treatment was necessary.

② Methyl mercaptan

A removing rate of more than 90% was obtained by using any of the three methods. In some cases, the removing rate was below 30% in the pre-treatment, but nevertheless, a rate of more than 80% was obtained.

③ Methyl sulfide

The removing rate of methyl sulfide fluctuated between 36% and 95% because its original concentration was close to the recognition threshold value. The effect was unstable because neither the pre-treatment nor the after-treatment shows any marked effect.

④ Dimethyl disulfide

Although the original odor concentration was close to the detective threshold value, the removing rate by using [Imp. carbon] + [Carbon], was more than 80% in the pre-treatment and 94 to 100% in the after-treatment for the original concentration. On the other hand, in the cases of [Chemical] + [Carbon] and [Water] + [Ozone] + [Carbon], the removing rate showed unstable values, i.e. between 38 to 100%.

⑤ Ammonia

Even by using any of these three methods, a good removal cannot be expected for ammonia since the removing rate was very low both in the pre- and after-treatments.

⑥ Trimethyl amine

The removing rate of trimethyl amine was about 50% in the pre-treatment and 94 to 99% in the after-treatment for the original concentration at the Chubu Sewage Treatment Plant.

d. Characteristics of deodorization effect

① Good deodorization effect was obtained for hydrogen sulfide and methyl mercaptan which are the major substances comprising offensive odor. The effect of pre-treatment was best in [Imp. carbon] + [Carbon] followed by the [Chemical] + [Carbon] and [Water] + [Ozone] + [Carbon] ranked last.

② By using sodium hypo-chrolite in [Chemical] + [Carbon], the removing rate in the pre-treatment was much improved.

2.5.3 Operation Controlability

An automatic control is possible in each deodorization method. However, in [Chemical] + [Carbon] and [Water] + [Ozone] + [Carbon], operation procedure becomes complicated when compared with [Imp. carbon] + [Carbon] because more auxiliary equipment is required.

The control of equipment when the contents of offensive odor substances vary greatly, as in case of [Chemical] + [Carbon], a constant deodorization effect can be obtained by adjusting the operation of the chemical pump.

It is difficult to control the ozone feeding rate in the case of [Water] + [Ozone] + [Carbon]. No control is necessary for [Imp. carbon] + [Carbon].

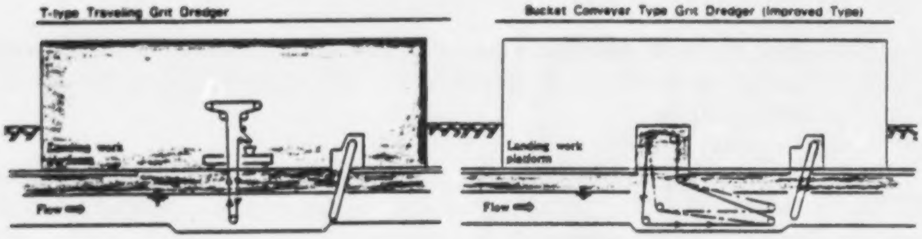
2.6 Future Problems

a. Enclosure of offensive odor source

In order to carry out effectively and economically the countermeasures against offensive odor, it is necessary to enclose and cover the location from which the offensive odor is generated and to collect securely a small amount of high odor unit gas. To achieve this, it is preferable to contain the main part of the grit dredger below the landing work platform and only treat the deodorization for the exhaust from the limited portion of the lower part.

Machines which do not protrude so much above the landing work platform, such as the grit dredger of bucket conveyor type, are preferable. However, in the grit chamber of the combined sewerage system, the usual fixed bucket conveyor type cannot be used because a large amount of grit enters in stormy weather. The city is developing and constructing a new type of equipment in which the sprochet wheel at the bottom can be raised or lowered (See Fig. 16).

Fig. 16 Types of Grit Dredger


T-type Traveling Grit Dredger
Landing work platform
Flow
Bucket Conveyor Type Grit Dredger (Improved Type)
Landing work platform
Flow

By using this equipment, when the lower part of the landing work platform is deodorized, the space volume will be decreased to about one fifth, such as at the Chubu Sewage Treatment Plant. A decrease in the capacity of deodorization equipment can thus be obtained.

b. Operation type with cost effectiveness

The odor unit of grit chamber is different according to the operating conditions i.e. whether the grit dredger is operating or not. In some cases when the grit dredger is not operating, no deodorization is necessary because the odor unit is at a low level. It is necessary to consider the operation method by which good cost performance is obtained for deodorization equipment since the stopping time is longer than the operating time.

c. Substances to be deodorized

The offensive odor generated from the grit chamber can be decreased by removing the offensive odor prescribed by law. However, in order to promote the deodorization more effectively, it is necessary to investigate the characteristic odor from sewage and establish countermeasures.

d. Odor unit of exhaust

The target value of odor unit at the stack outlet was set at 30 in this experiment so as the exhaust to be odorless at the boundary line of site, but it is necessary to review to what extent the odor unit at the stack outlet is allowed in order to meet with future environmental circumstances.

e. Odor control for the grit chamber of sanitary sewerage system

In this experiment, the grit chamber of combined sewerage system was used, but in future it will be necessary to investigate the actual operating conditions in a sanitary sewerage grit chamber and establish practical countermeasure for it.

Afterword

The city decided to investigate the actual state of existing deodorization equipment in the sewage treatment plants and pumping stations, and to establish countermeasures for offensive odor. In this connection, deodorization experiments for low odor unit gas were first carried out at the grit chambers which had been installed at many locations. Some observations on the results have been described above.

In Yokohama, it has been considered necessary to place covers at the primary sedimentation tanks and also in some places where the surroundings warrant the placing of covers at aeration tanks. Continuing systematic reviews and development are considered necessary for offensive odor control and deodorization.

Finally, I would like express my sincere appreciation for the kind cooperation of Messrs. Y. Hattori, Senior Technical Advisor, M. Noguchi, S. Kaminaga, S. Kobayashi, S. Ushiyama, T. Kawabata, T. Yamada, H. Ishii, S. Yoshimi, and other personnel concerned.

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AUTOMATIC WATER QUALITY ANALYZERS FOR SEWERAGE SYSTEMS

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Public Works Research Institute
Ministry of Construction

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AUTOMATIC WATER QUALITY ANALYZERS FOR SEWERAGE SYSTEMS

CONTENTS

1. Introduction 245

2. Overview of the Use of Automatic Water Quality Analyzers in Japan 246

3. Automatic Analyzers for Monitoring Organic Loading in Treated Water 248

4. Automatic Analyzers for Hazardous Substances 254

5. Conclusion 261

1. Introduction

Automatic water quality analyzers are finding increasing use in sewerage systems, particularly at sewage treatment plants of large cities.

The strong interest by engineers in the quality control of inductrial waste water discharged into public sewerage systems and in the process control of sewage treatment plants has led to the utilization of automatic analyzers. The energy crisis has drown the attention of process engineers to automatic control in search of means of energy-saving. Moreover, the amendment of the Water Pollution Control Act in 1979 obliges all sewage treatment plants in areas to which the loading regulation system is applicable to carry out automatic measurement and recording of the concentrations of organic substances in treated water.

In these circumstances, the development and actual use of automatic water quality analyzers will be even further promoted.

This paper gives an overview of the use of automatic water quality analyzers in sewerage systems in Japan, the outlook of automatic analyzers for organic loading monitoring under the loading regulation system, and the current state of development of automatic monitoring equipment for hazardous substances.

2. Overview of the Use of Automatic Water Quality Analyzers in Japan

697 sewerage authorities throughout Japan were surveyed by questionnaire to grasp the current state of use of automatic water quality analyzers. Some of these authorities have no sewage treatment plants, and as few as 113 (16.2%) responded. On a population served basis, however, the ratio of respondents to the total was about 80%.

Fig. 1 summarizes the results of the questionnaire survey.
Automatic water quality analyzers in use number 19 types in all.
Fig. 1(a) gives the number of units of each analyzer in use.
The most popular instrument is the DO meter with 373 units is use,

followed by pH meters, sludge density meters, MLSS meters, thermometers, turbidimeters, sludge level guages, conductivity meters, residual chlorine meter, and so forth.

Fig. 1(b) shows analyzers classified by the location of measurement. DO meters, pH meters, sludge density meters, MLSS meters, and so forth, which lead the list of analyzers, are used chiefly for measuring water quality in treatment processes. These instruments are employed mostly for monitoring water quality. However, DO meters, MLSS meters, residual chlorine meters and sludge density meters, etc., are also used in some cases for automatic control of treatment processes. Application in automatic control is expected to increase in the future.

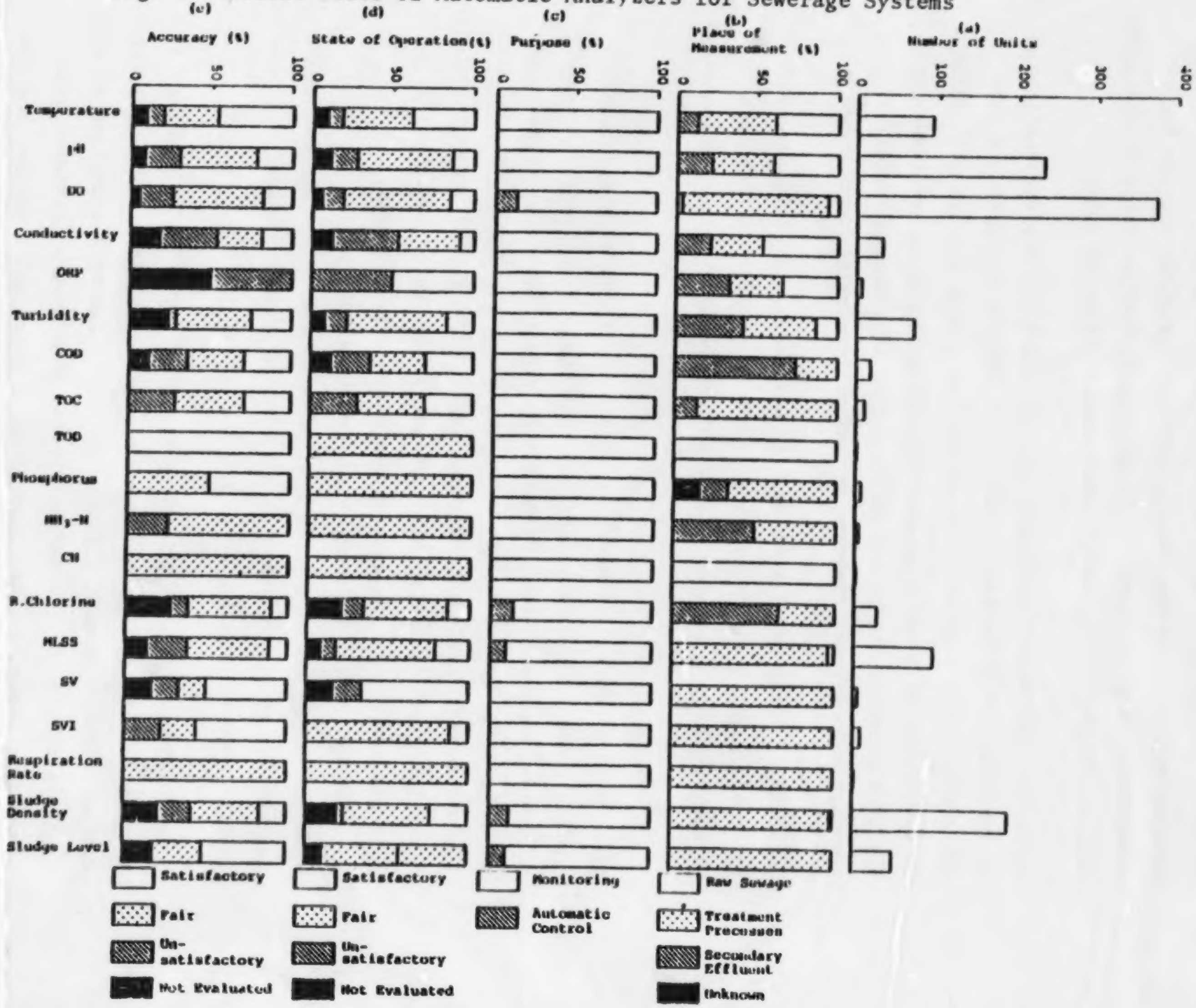
Many respondents reported that the majority of analyzers in use were satisfactory as far as operation of the instruments was concerned. It is rather surprising, however, that the performances of many conductivity meters and ORP meters, which are expected to have less problems, were reported to be unsatisfactory.

This may be because the meaning of these parameters is not clear, resulting in poor maintenance of the instruments. Other instruments with which "unsatisfactory in operation" accounted for a considerable proportion of the total included COD meters, TOC meters, SV meters, and pH meters.

The parameters for which many respondents considered the accuracy to be unsatisfactory included ORP, conductivity, TOC, COD, pH, ammoniacal nitrogen, and MLSS.

It is noteworthy that those instruments which were reportedly poor in operation were generally viewed as having poor accuracy. Dissatisfaction with accuracy was generally stronger than with operation.

Fig. 1 Current state of Automatic Analyzers for Sewerage Systems



3. Automatic Analyzers for Monitoring Organic Loading in Treated Water

The Water Pollution Control Act was amended in June 1979 to add a loading regulation system to the effluent standards then in force (concentration regulation) in an attempt to further restrict the loadings of pollutants descharged into public bodies of water.

Currently, the Seto Inland Sea, Ise Bay and Tokyo Bay are designated as specified water bodies to which this loading regulation system is applicable. The loading to be controlled is the COD loading, which is stipulated as a manually analyzed COD measured in accordance with the acid permanganate method as specified in Japanese Industrial Standard K-0102-1974, "Testing Method of Industrial Waste Water."

Under the loading regulation system, polluters, including sewage treatment plants, are required to limit their effluent loadings to within respective assigned values and at the same time to measure and record them.

Sources whose daily discharge is in excess of 400 m^3 are obliged to install a continuous flowmeter and an automatic analyzer for organic substances that will provide data with a significant correlation to manually analyzed COD values. The analyzers recommended for this include the COD analyzer, VU photometer, TOC analyzer and TOD analyzer. Installation of automatic monitoring instruments is mandatory by June 1981 for every sewage treatment in specified areas.
In order to obtain information for selecting analyzers, problems with the operation and maintenance of instruments now in use and the correlation between automatic analysis and manual COD analysis were investigated in detail.

As of December 1979, 15 COD analyzers were in use at 14 sewerage systems. Of these, 13 units were installed for monitoring effluent COD at sewage treatment plants. The other two were installed at pump stations to monitor raw sewage flowing into sewage treatment plants. The oldest instrument had been in use 5 years and 4 months, and the newest 1 month; the average age being about 2 years. In most cases, the cleaning and calibration of the instruments was carried out weekly.

Table 1 lists results for the ratio of the period during which the instruments were actually operated to the total intended period of operation (time ratio of operation). Periods during which the instruments were stopped for regular maintenance or because of power failure are included in the period of actual operation. As shown in the table, the time ratio of operation very high. About half of instruments were actually operated for more than 90% of the intended period. It should be noted however that instruments handling raw sewage showed a poor time ratio of operation of 50% or lower. The time ratio of operation was higher when the instruments were maintained by their manufacturers than by employees of the authorities, suggesting that maintenance of these instruments requires relatively high skill.

Troubles reported in sampling systems were clogging of pumps and sampling tubes by suspended solids, or development of slime. Analyzers suffered from clogging of the potassium permanganate titrator, short life of the reference electrode, clogging of drain pipe by deposition of silver chloride, and corrosion of parts. These facts show that the instruments themselves need to be greatly improved. The correlation coefficients with manufally analyzed COD were as given in Table 2 and show a similar tender to the time ratios of operation. It is quite logical to consider that well-maintained instruments produce data that closely conform with manually analyzed COD data. The two instruments which handled raw sewage showed low correlation coefficients; maintenance alone may not be responsible, but account should be taken of the fact that dilution of samples cannot be varied in automatic analyzers.

As of December 1979, 9 sewage treatment plants were using 10 TOC analyzers for continuous monitoring of TOC. The oldest instrument had been in operation 6 years and 9 months, and the newest 2 months; the average being about 2 years. Of these, 7 units were used for monitoring effluent TOC, and the other 3 for automatic TOC measurement of raw sewage or primary settled sewage.

Some instruments had already been disqualified from automatic continuous measurement, and only five installations could offer data on the time ratio of operation. (Table 3)

Table 1 Ratio of the Period in which COD Analyzers were Actually Operated to the Whole Period

Sample	Maintenance by:	Ratio of Actually Operated Period (%)			
		more than 90	70 ∿ 90	50 ∿ 70	Less than 50
		Number of COD Analyzers			
Raw Sewage	Sewerage Authority				2
Secondary Effluent	Sewerage Authority	2	3		2
	Manufacturer	5	1		

Table 2 Correlation Coefficient between COD Data by Manual Analysis and Automatic Analyzer

Sample	Maintenance by:	Correlation Coefficient			
		more than 0.9	0.7∿0.9	0.5∿0.7	Less than 0.5
		Number of COD Analyzers			
Raw Sewage	Sewerage Authority				2
Secondary Effluent	Sewerage Authority	2	2		2
	Manufacturer	3	2		

Table 3 Ratio of the Period in which TOC Analyzers were Actually Operated to the Whole Period

Sample	Ratio of Actually Operated Period (%)			
	more than 90	70 ∿ 90	50 ∿ 70	less than 50
	Number of TOC Analyzers			
Raw Sewage		2		1
Secondary Effluent	1	2		1

Except for one case, all these instruments were maintained by employees of the sewerage authorities. Half of the instruments were washed once a day, and many users reported that necessity of freaquent maintenance was one of the demerits of TOC analyzers. Troubles reported with TOC analyzers include clogging of the sampling tube, the furnace corrosion, entrainment of moisture into the infrared analyzer, and large fluctuations in the flow of carrier gas. In principle, clogging and corrosion problems are difficult to overcome.

There was only a single case in which a TOD analyzer was employed for automatic measurement; it had been used for 3 years and 6 months for measurement of raw sewage with a high time ratio of operation of 85%. The instrument was maintained daily, which is a considerable amount of work.

Recently, the UV photometer has been attracting the attention of those concerned because of its simple maintenance. At present, there are 4 sewage treatment plants where UV photometers are employed for monitoring organic substances in effluent. The oldest instrument is 9 months old, and the newest is 1 month old. In almost all cases, the time ratios of operation were nearly 100%. Troubles reported include contamination of the flow cell due to insufficient automatic cleaning, drift in measurements due to changes in ambient temperature, and scattering of measurements due to large changes in turbidity. Although the UV photometers used have the function to concurrently measure the absorption of visible light (the spectral sensitivity differs with manufacturers) in order to compensate the effect of turbidity; there may be such instruments that need improvement in correction mechanism.

It is well known that there usually a good correlation between UV absorbance and COD, but it is also known that the degree of correlation varies with the sewage and sometimes varies considerably with time even at the same treatment plant. The eligibility of the UV photometer as practical and effective instrument for monitoring COD loading will depend largely on the stability of the correlation.

The UV photometer is essentially an instrument to measure dissolved organic matter. It is not suitable for use where suspended solids variation is large.

Apart from the survey referred to above, the Sewerage Bureau of the Osaka Municipal Government carried out a comparative field test of COD analyzers of five manufacturers and UV photometers of three manufacturers using secondary effluents for about two months to examine the performance of these instruments as automatic monitoring instruments under the COD loading control program. During the test, the instruments were maintained by their respective manufacturers once a week. To investigate the correlation with manually analyzed COD, 24 hour-round-the-clock surveys were carried out three times in addition to regular sampling and analysis at fixed time every day throughout the test period.

Most of the instruments operated satisfactory. The lowest time ratio of operation of the COD analyzers was 86% and the highest was 100%. The time ratios of operation of the UV photometers were generally higher than those of the COD analyzers. The lowest time ratio of operation of the UV photometers was 96%.

Troubles experienced with the COD analyzers during the test period were blocking of the potassium permanganate titrator and clogging of the drain pipe of the reactor vessel with deposited silver chloride. UV absorption measurements in some cases were impaired by base line drifts which may have been attributable to insufficient automatic cleaning of flow cells. The correlation coefficients determined from each round-the-clock survey were all above 0.8, but for all the data during the test period, the overall correlation coefficients were lower. COD analyzers and UV photometers are considered to be better instruments than TOC or TOD analyzers as automatic monitoring equipment required by the COD loading control system. However, performances of some models of these instruments are not necessarily satisfactory. Therefore, a standard specification for these instruments is to be prepared in the near future.

4. Automatic analyzers for hazardous substances

Although the necessity for automatic monitoring of hazardous substances is very high, actual application is far from wide spread due to the lack of suitable automatic analyzers and for other reasons.

At present, except for indirect monitoring by the measurement of pH, electrical conductivity, and so forth, there is only one case of automatic monitoring of cyanide in raw sewage.

Concerned over the situation, the Ministry of Construction has commissioned the Association of Electrical Engineers to develop automatic analysis and monitoring instruments for hazardous substances. The Association of Electrical Engineers has organized a committee of users and manufacturers to evaluate and modify existing instruments and to develop new devices through laboratory and field tests with the assistance of manufacturers. Up to now, total cyanide, chromium, cadmium, copper, lead and mercury have been studied. The development of instruments for total cyanide has been completed, and their standard specifications prepared. For chromium, cadmium and copper, one-year field tests using prototype models have just been completed, and standard specifications for these instruments are being prepared. Surveys on lead and mercury analyzers are still in the first stages.

Except for the mercury analyzer, all the instruments investigated by the Committee use automated colorimetry or an ion selective electrode system. Atomic absorption spectrophotometry, high-frequency plasma spectrography, X-ray fluorometry, and other highly sophisticated methods may be utilized, but their applications have not yet been studied since this would require a change in the concept of automatic analyzers for on-site use. The work being done by the committee is summarized below. A description of cyanide analyzers is omitted here because they were described at the fifth and the sixth conferences.

Total chromium analyzers

Total chromium analyzers using the diphenylcarbazide method are now available from several manufacturers. Their applicability to the automatic monitoring of chromium in raw sewage has been tested, and it was found that they would fail where the concentration of reducing substances was high or changeable over a wide range. Laboratory tests were then conducted to look for possible improvements. It was found that preliminary oxidation of samples by aeration under acid conditions before adding potassium permanganate was effective in reducing and stabilizing the consumption of potassium permanganate.

The use of Sodium hypobromide, or other oxidizing agents, which can easily remove excessively dosed chemicals were studied, but their application to automatic analyzers was throught difficult because of corrosion, reagent stability, and so on.

An improved model, a batch type device with a preoxidation stage and an automatic background compensating function was manufactured. This was tested for six months at a treatment plant handling mainly metal finishing wastewater and for another six months at a municipal sewage treatment plant.

Clogging of the drain pipe from the heating vessel, crystallization of diphenylcarbazide in the pipe during winter, and other mechanical troubles were encountered at the initial test. The device has been running fairly well since these troubles were overcome. Data from manual analysis and the data from the automatic analyzer are compared in Fig. 2.

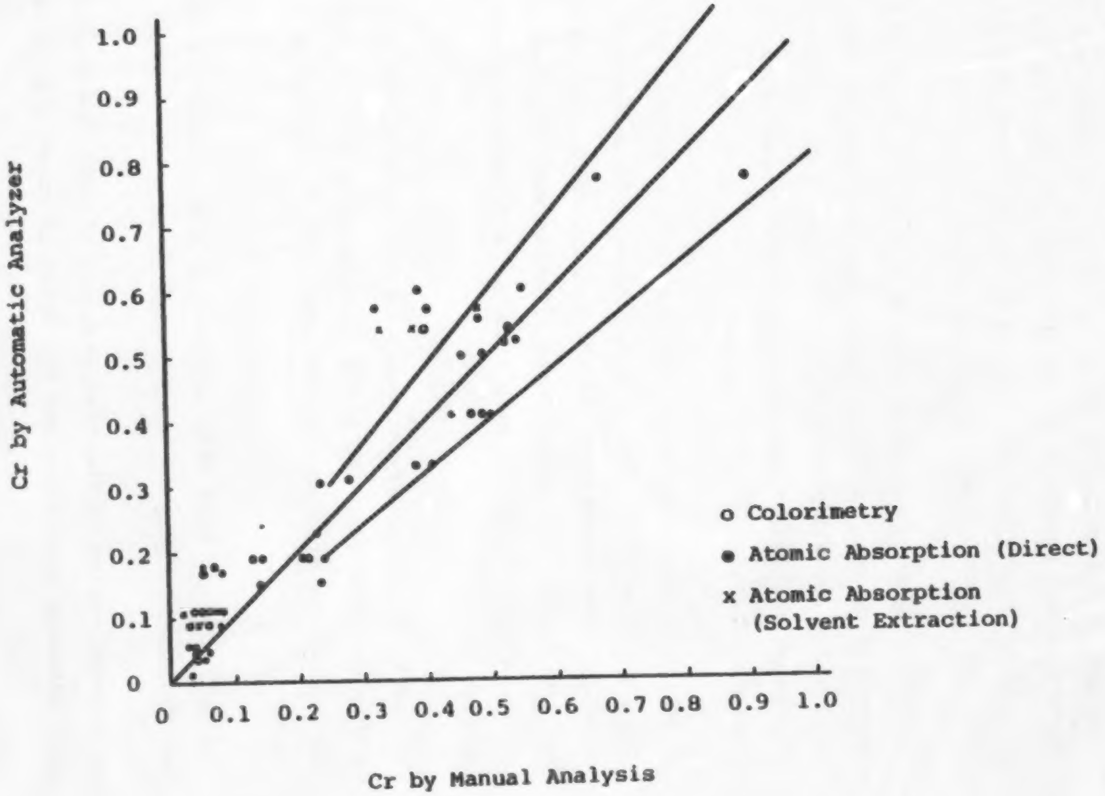
Although the degree of agreement is not so good, the bulk of data fall within an error range of ± 20%. Calibration was not carried out throughout the most part of the test period to examine full-scale drift.

The accuracy may have been improved if the calibration had been carried out frequently, that is, once a week or so.

At present, based on the test results, efforts are being made to prepare standard specifications for total chromium analyzers.

For automatic analysis of hexa-valent chromium, there are several instruments available on the market, which also use the diphenylcarbazide method. Their application to raw sewage is problematic from the viewpoint of reliability because the automatic background correction range is often exceeded by sample blanks and because the presence of reducing substances of high concentrations greatly interferes with

Fig. 2 Comparison of Analytical Values of Cr by Automatic Analyzer and Manual Analysis



the diphenylcarbazide method.

For this reason, an analyzer based on UV absorption has been studied. This method, however, has problems. At around 365 nm where UV absorption by hexa-valent chromium has its peak, interference from iron is considerable. At a longer wavelength, for instance 436 nm, the iron interference decreases, but the absorption sensitivity is lowered, and at the same time absorption by organic substances becomes relatively significant. It was found through laboratory and field tests that these problems were difficult to overcome.
Therefore, UV absorption is judged impractical.

Cadmium analyzers

There is no colorimetric method suitable for automatic measurement of cadmium. Therefore, the development of an automatic analyzer using an ion selective electrode has been proceeded with. So far, there have been two new developments in iron selective electrodes. One is a ceramic electrode whose surfaces can always be maintained in good condition by means of a rotor turned by a magnetic stirrer. This electrode minimizes drift during continuous long-term operation. Another is the development of a new reference electrode that responds to organic substances just as the cadmium ion selective electrode does with its potential not being affected by cadmium.

An automatic analyzer using these electrodes was made and tested. Its operation is as follows. A measured sample is acidified to become 1.2N hydrochloric acid solution, and then heated to dissolve cadmium in the solids, and filtered through a coarse media filter. As masking agents for ions of iron, copper and lead, salicylic acid and thiourea are added to the filtrate, then the fittrate is passed through an anion exchange resin column to adsorb cadmium. Cadmium is desorbed with distilled water, and the eluate is adjusted to a proper ionic strength and measuredments made with ion selective electrodes. Two anion exchange columns are provided, which are used alternately to minimize contamination.

Field tests were carried out for six months at each of two municipal sewage treatment plants. At first, troubles were caused by deterioration of tubes and tube fittings by hydrochloric acid. After the tube materials were improved, the analyzer operated in good order. Fig. 3 compares data from manual analysis and from the automatic analyzer.

As shown, cadmium could measured to an accuracy of ±0.1 pCd even when the concentration was as low as about 0.002 mg/ℓ.
At present, standard specifications for automatic measuring equipment based on this system are being prepared.

Copper analyzer

The colorimetric analysis of copper (cuprous ion) based on the bathocuproine method is less affected by interferences and is suitable for automation. A batch type measuring instrument with an automatic sample blank compensating function was subjected to field tests.

Initially, a pre-treatment section was used consisting of: (1) acid addition and heating to dissolve copper contained in solids; (2) filtration through a coarse media filter; (3) pH adjustment; and (4) addition of ascorbic acid to reduce the cupric ions.

The field tests revealed that the measurements by the automatic analyzer always gave smaller values than manual analysis. The cause of this was attributed to the facts that when the concentration of sulfides in the sample was high, the copper dissolved by the added acid formed copper sulfides again, when the pH was adjusted before color development. For this reason, the instrument was modified to add potassium permanganate to oxidize the sulfides before filtration.
As a result of the modification, it became necessary to add ascorbic acid prior to filtration to prevent such troubles as clogging of the filter and copper adsorption due to the formation of manganese dioxide.

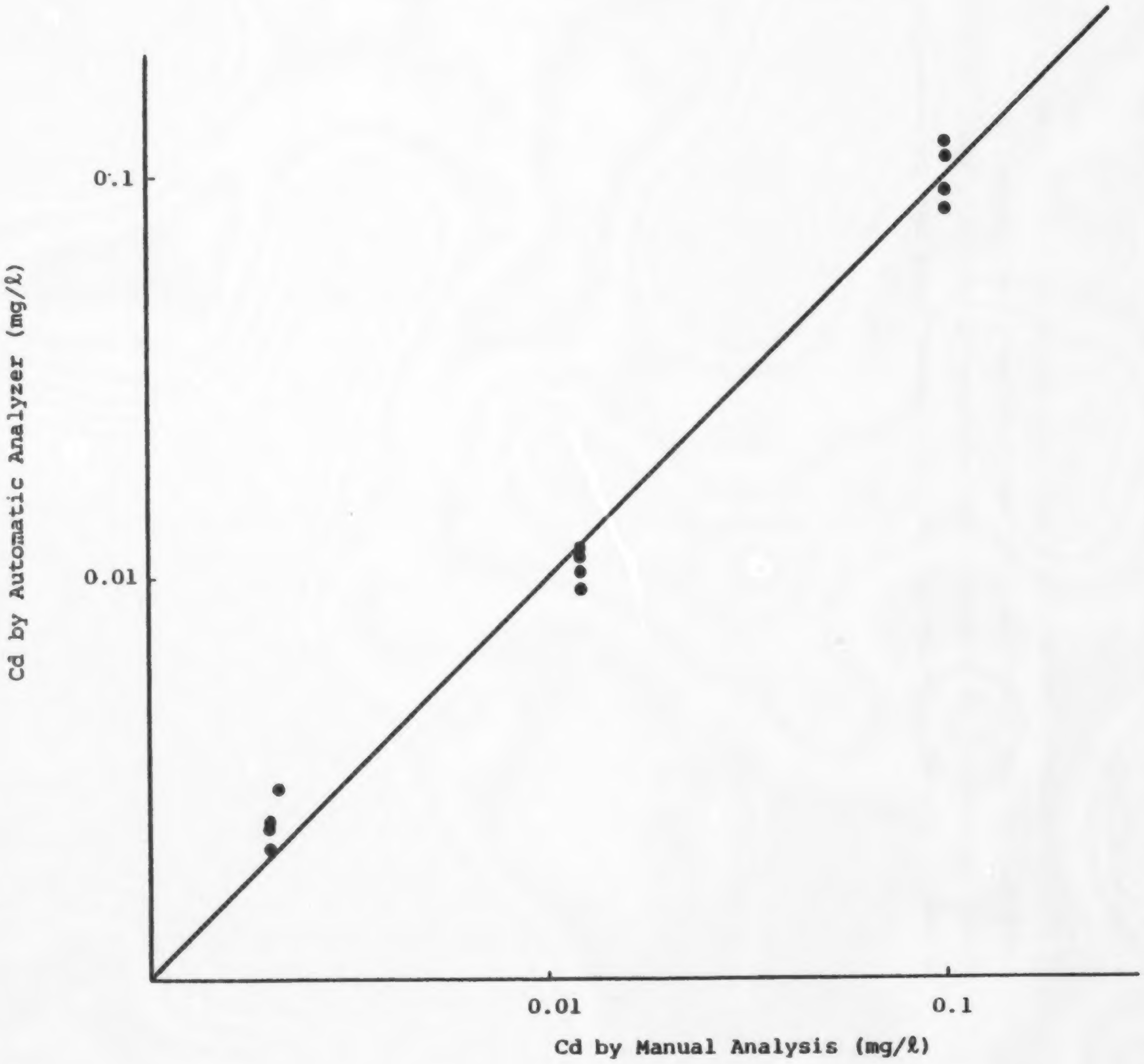
At present, standard specifications are being prepared for copper analyzers using this modified system.

Lead analyzers and mercury analyzers

Laboratory tests are under way to develop lead and mercury analyzers. Field tests will be started in FY1980.

For the automatic measurement of lead, an improved ion selective electrode has already been developed just as in the case of cadmium,

Fig. 3 Comparison of Analytical Values of Cd by Automatic Analyzer and Manual Analysys



and an automatic analyzer using it is under study.

Like cadmium, lead forms a negatively charge chloride ion complex, and can therefore be separated in an anion exchanger. It is expected that a system similar to the cadmium analyzer will be applicable to the analysis of lead.

As regards mercury, two manufacturers have already put on sale automatic inorganic mercury analyzers which use flameless atomic absorption spectrophotometry. At present, these instruments are being laboratory tested to develop a pretreatment system allowing total mercury analysis.

5. Conclusion

Automatic water quality measurement in the field of sewerage has two applications; one for the control of treatment processes, and the other for monitoring qualities of inflow sewage and effluent.

Automatic analyzers and sensors for the control of treatment processees, particularly the activated sludge are at the stage of practical use, except those for measuring organic loadings.

DO meters, pH meters, MLSS meters, and, sludge density meters, for example, are widely used. They will be used increasingly in keeping with the development and spread of automatic treatment processes control.

As regards automatic analyzers for monitoring influent sewage, very few kinds of analyzers, such as pH meters, are currently in use. Automatic analyzers for monitoring toxic substances in raw sewage are being developed, and those for cyanide, total chromium, cadmium and copper have now become available. But they must be still further improved as they are costly and require fairly heavy maintenance.

As regards the automatic monitoring of effluent, COD analyzers and UV photometers will find increased use since the amendment of the Water Pollution Control Act makes it mandatory for the sewerage authorities in specified areas to carry out automatic measurement of COD loading in effluent.

Seventh US/JAPAN Conference
on
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CURRENT STATUS OF COMBINED SEWER PROBLEMS AND THEIR CONTROL MEASURES IN JAPAN

May 21, 1980
Tokyo, Japan

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CONTENTS

PAGE

1. Introduction 265

2. The score of combined sewer problems 267

  2.1 General 267

  2.2 Characteristics of combined sewage 271

  2.3 Mathematical models 281

3. Measures taken against combined sewer problems 291

  3.1 Improvement of intercepting systems 291

  3.2 Storage 294

  3.3 Experimental measures 304

4. Conclusions 309

1. Introduction

Combined sewer systems in which sanitary sewage and stormwater are collected and disposed of through a common sewer have lower construction costs than separate sewer systems. They are widely employed in Japan because, in addition to being inexpensive, they are easy to install along the narrow roads in urban areas. Another advantage of combined sewer systems is their simplicity in that the collection and disposal of sewage and inundation control can be achieved by a single network of sewers.

The history of sewerage, starting with primitive drains and developing to modern urban drainage systems terminating in treatment plants, has seen the spontaneous growth of combined sewer systems. In old cities in Europe and North America, and in Tokyo and Osaka in Japan, where sewerage has a long history, combined sewer systems are prevalent.

In combined sewer systems, however, when it rains, part of the sewage is sometimes directly discharged into receiving waters without treatment. In response to public demand for a higher-quality water environment, it has recently been recognized that control of combined sewer overflows is of increasing importance.

When the inflow to a sewage treatment plant is increased by stormwater, part of the sewage is discharged after only sedimentation and disinfection. Some experts warn that this kind of primary effluent is a source of water pollution that cannot be reglected.

In Japan, with the exception of the metropolises, most local cities lag far behind in terms of the coverage of sewerage. National policies for sewage works have yet to place major emphasis on the extension of sewers and new installation and expansion of sewage treatment plants.

There are few, if any, cities that can afford to study the problems inherent in the combined sewer system, and provide measures against them.

Unlike sewage treatment plants which are local in scale, the sewer system is larger in size running under roads in complicated meshes.

Therefore, the improvement of sewer system involves many problems both technically and financially. And, this has delayed the solution of combined sewer problems.

A recent survey has warned that not only the overflows from the combined sewer, but stormwater runoff from highly developed areas is fairly contaminated. The problem here lies not only in pollution due to combined sewer overflows, but shows a more complex spectrum involved in all discharge from urban drainage systems and their relations with the quality of receiving waters.

This paper deals with combined sewer problems that have been brought to the fore in Japan, and the measures that have been, and will be, taken.

2. The score of combined sewer problems

2.1 General

As of the end of fiscal 1979, municipalities with public sewage works numbered 579 in Japan. Most of them constructed their sewage works in the past 10 years or so. Municipalities which are experiencing serious combined sewer problems are in the minority. Water pollution problems in Japan today are still largely accounted for by dry weather sanitary sewage in the districts without sewerage.

Combined sewer problems of sewage collection and disposal have been subject to wide and careful attention by sanitary experts.

Combined sewer problems are, for instance, always on the top of the list of topics to be discussed at meetings of the Workshop on Sanitary Works, a sanitary engineer forum. These discussions may be summarized into the following three points.

(1) There are cases of dry weather overflows from diversion chambers where the volume of sewage generated per area has been increased by the increase in per capita water consumption, by increase in population in areas with sewers, or by urban renewal.

(2) Where wet weather effluent from sewage treatment plants, diversion chambers and pump stations is a major pollutant source for receiving waters, the quality of receiving waters can no longer be improved substantially as long as existing sewerage systems remain unchanged.

(3) The separation of sewers involves both technical and financial difficulties.
Where the stormwater runoff itself is contaminated, the separation of sewers does not offer any viable solution to the problem.

Although these discussions have not yet been revolved, almost every city has come to employ a separate sewer system for its new installations based on the situation in Europe and United States and of the role played by sewerage in the conservation of the quality of receiving waters.

This is not because they regard the separate sewer system as the solution to wet weather problems, but because the separate sewer system is considered to be more economical than the combined sewer system where the existing drainage channels can be turned to good account in the drainage of stormwater.

In this case, water pollution by dry weather sewage can be solved more rapidly by the separate system than by the combined sewer system.

According to a survey conducted in fiscal 1973, about 2,080 km^2 area (27% of the sewerage projected area of 7,732 km^2) had sewer construction completed, which included 1,546 km^2 of combined sewer areas and 540 km^2 of separate sewer areas. At the time when the survey was conducted, 76% of the area with sewers was accounted for by combined sewer systems, but by the end of the sewerage construction project, that percentage will be reduced to 36% (see Fig. 1a).

Of the population projected of 70.3 million to be served by sewerage, about 38 million, or 54%, had already been covered with sewers, and 69% of this percentage live in areas with combined sewer systems. In the final stage of the project, the population with combined sewer systems will be 53% of the projected population (see Fig. 1b), exceeding the population covered by the separate sewer systems. Notwithstanding on-going projects to promote separate sewer systems, the problems of combined sewer systems will remain undiminished.

The overflows from the combined sewer system are under the control of uniform effluent standards pursuant to the Water Pollution Control Act which governs general discharges. (Table 1)

Some municipalities have regulations that impose tighter control over effluents than the Government-set uniform effluent standards.

The improvement of combined sewer systems to meet such tightened standards is not only unrealistic, but is also unlikely to improve the water quality of the receiving waters.

At present, the Japan Sewage Works Association has an in-house committee to study how and to what extent the combined sewer system should be improved.

Fig. 1a Sewerage Projected Area

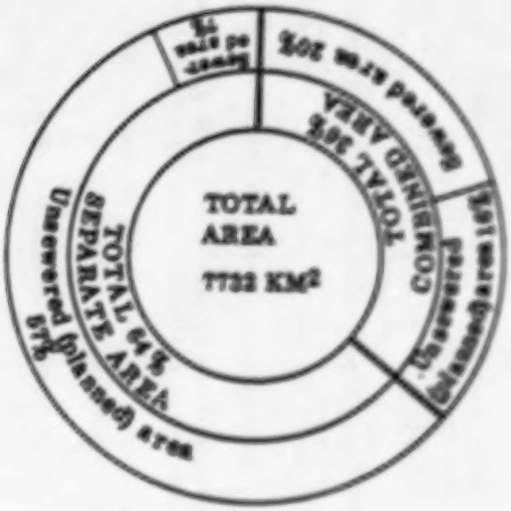


Fig. 1b Sewerage Projected Population

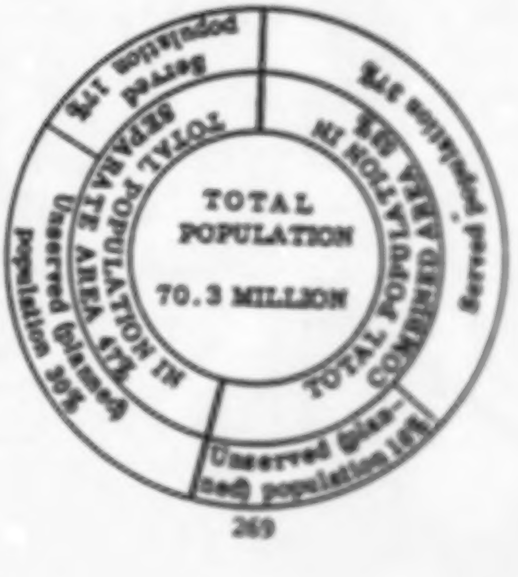


Table 1. Uniform Effluent Standards

(1) Items for the protection of the human health

Item	Cyanide	Alkyl mercury	Organic phosphorous	Cadmium	Lead	Chromium (hexavalent)	Arsenic	Total Mercury
Standard value	Maximum	Not detected	Maximum	Maximum	Maximum	Maximum	Maximum	Not detected
	1 ppm		1 ppm	0.1 ppm	1 ppm	0.5 ppm	0.5 ppm	

(2) Items for the conservation of the living environment (General items)

Item	pH		BOD		COD (Mn)		SS		No. of coliform groups
Standard value	Rivers & lakes	Sea	Daily average	Maximum	Daily average	Maximum	Daily average	Maximum	Daily average
	5.8-8.6	5-9	120 ppm	160 ppm	120 ppm	160 ppm	150 ppm	200 ppm	3,000 per cm^3

Remarks: 1. The above effluent standards is to be applied to (water emitted from) plants or factories with the normal daily discharge of more than 50 cubic meters.

2. The COD standards are not to be applied to water discharged directly into rivers from plants or factories, and the BOD standards are not to be applied to water discharged directly to lakes or sea from plants or factories.

3. The pH standards are not to be applied to (water discharged from) the sulphur mining industry (including the mining of iron sulfide ores co-existing with sulphur).

(3) Items for the conservation of the living environment (Special items)

Item	Oil (substances of extract N-hexane)		Phenols	Copper	Zinc	Iron (soluble)	Manganese	Chromium	Flourine
	Petroleum & related oils	greases							
Standard value	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum	Maximum
	5 ppm	30 ppm	5 ppm	3 ppm	5 ppm	10 ppm	10 ppm	2 ppm	15 ppm

Remarks: 1. The above effluent standards are to be applied to (water emitted from) plants or factories with the normal daily discharge of more than 50 cubic meters.

2. The iron (soluble) standards are not to be applied to (water emitted from) the sulphur mining industry (including the mining of iron sulfide ores coexisting with sulphur).

2.2 Characteristics of combined sewage

How much pollutant load is discharged from the combined sewerage during wet weather? What ratio does it contribute to the total load as against secondary effluents from sewage treatment plants? These are the first questions to be asked in clearing up the problems of combined sewer systems. In line with these questions, 16 experimental districts were assigned throughout Japan, as shown in Fig. 2, for the purpose of investigating the quantity and quality of dry-weather and wet-weather combined sewage. A survey was conducted for 3 years from fiscal 1975 to fiscal 1977. Of the 16 experimental districts, 4 had separate sewer systems, for which the quantity and quality of stormwater runoff was examined.

The characteristics of each experimental district are summarized in Tables 2a and 2b.

It is often said that urbanization has inflicted serious effects on the hydrological cycle in areas in which urbanization has taken place. The problem of urban drainage systems in wet-weather conditions may be regarded as one problem of the hydrological cycle in urbanized areas.

Fig. 3 shows the relationship between the peak dry-weather flow and population density in the drainage area. Urban renewal, or an acromegalic state, as represented by the mushrooming of high-rise buildings replacing low-rise houses is often seen in built-up areas.

This counts a snapback in daytime population, increasing the dry weather peak flow.

In Japan, side weirs are widely used as a diversion facility, and the dry-weather overflow is in many cases caused not by the malfunction of diversion facilities, but as a result of increase in peak dry-weather flow that exceeds the capacity of interceptors.

Figs. 4a, 4b and 4c show the relationships between total rainfall and total stormwater runoff in three areas having different ratios of impervious area to total area. In the figures, the slopes of the solid lines are set equal to the ratio of impervious area. Rainfall

Fig. 2 Map of The Experimental Districts

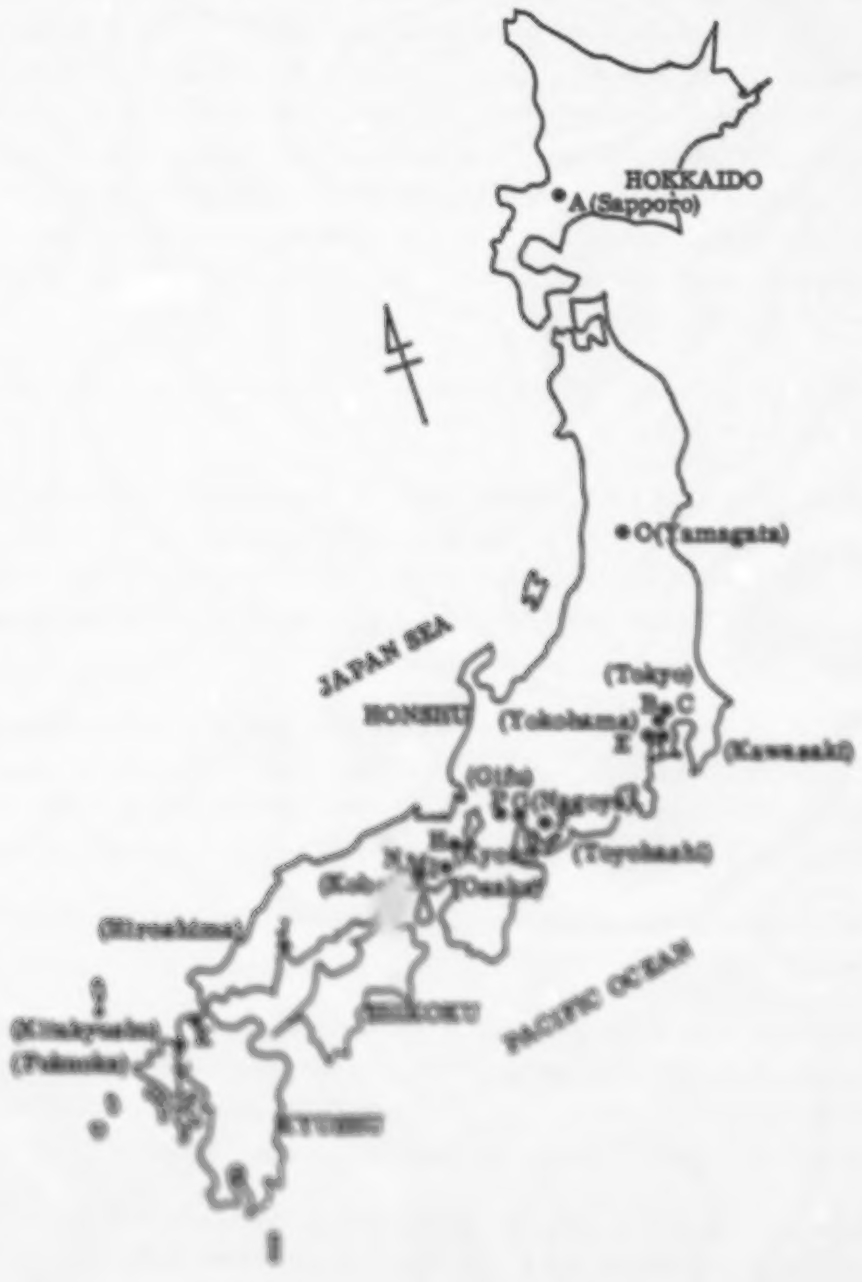

HOKKAIDO
A(Sapporo)
O(Yamagata)
JAPAN SEA
HONSHU
(Tokyo)
B
C
(Yokohama)
E
(Kawasaki)
(Gifu)
F
G
(Nagoya)
H
(Kyoto)
(Toyohashi)
N
(Kobe)
(Osaka)
(Hiroshima)
SHIKOKU
PACIFIC OCEAN
(Kitakyushu)
(Fukuoka)
KYUSHU

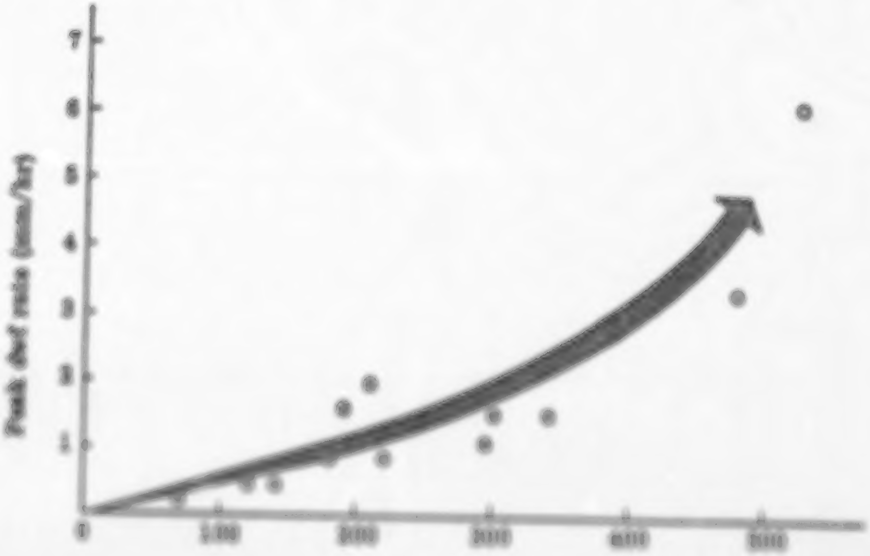
Table [illegible] Characteristics of Experimental Districts (Combined Sewer System)

Drainage district	Drainage system	Drainage area (ha)	Population		Sewer diameter (mm)	Max. flow rate in dry-weather (m^3/S)	Land use						Road area		Impervious area	
			Daytime	Resident			Residential (ha)	(%)	Commercial (ha)	(%)	Industrial (ha)	(%)	(ha)	(%)	(ha)	(%)
[illegible]	Pumping	44.33	6,655	4,016	300~900	0.057	44.33	100	0	0	0	0	6.95	15.7	15.27	34.4
[illegible]	Gravity flow	542	165,019	155,091	6000x4325	2.11	397.5	73.5	68.29	12.6	74.80	13.9	116.6	21.6	288.0	53.2
[illegible]	Pumping	269.5	50,690	61,167	6000x3600	1.57	16.79	6.2	46.84	17.4	205.87	76.4	3.82	1.4	186.3	69.1
[illegible]	Pumping	35.13	12,298	9,003	1650x1650	0.10	21.45	61.1	13.68	38.9	0	0	4.46	12.7	16.08	45.8
[illegible]	Gravity flow	22.08	3,570	4,220	250~1200	0.052	21.36	96.8	0.71	3.2	0	0	2.39	10.8	7.68	34.8
[illegible]	Gravity flow	68.37	38,260	8,718	230~1950x1950	0.282	0	0	68.37	100	0	0	21,05	30.8	53.33	78.0
[illegible]	Gravity flow	39.5	12,800	1,780	250~1650	0.109	0	0	39.5	100	0	0	9.80	24.8	26.07	66.0
[illegible]	Gravity flow	148.40	37,180	25,260	250~3000x1930	0.809	32.10	21.6	32.86	22.1	83.53	56.3	31.27	21.1	114.46	77.1
[illegible]	Pumping	45.50	63,200	804	300~2000x1350	0.385	0	0	45.50	100	0	0	24.3	53.4	39.1	85.9
[illegible]	Pumping	215.16	27,000	36,727	250~2200x2200	0.261	108.54	50.5	41.6	19.3	65.0	30.2	44.77	20.8	101.11	47.0
[illegible]	Gravity flow	57.6	3,975	3,063	200~1200x1600	0.028	57.6	100	0	0	0	0	7.0	12.2	15.0	26.0
[illegible]	Gravity flow	17.61	9,700 (day) 17,000 (night)	3,098	230~1360x1060	0.247	0	0	17.61	100	0	0	5.35	30.4	15.50	88.0

Table 2.b Characteristics of Experimental Districs (Separate Sewer System)

Drainage district	Drainage system	Drainage area (ha)	Population		Sewer diameter (mm)	Land use						Road area		Impervious area	
			Daytime	Resident		Residential (ha)	(%)	Commercial (ha)	(%)	Industrial (ha)	(%)	(ha)	(%)	(ha)	(%)
M	Gravity flow	17.17	9,300	2,695	1200x 1030~ 30x70	8.55	49.8	8.02	46.7	0.60	3.5	5.72	33.3	12.10	70.5
N	Gravity flow	26.75	2,420	3,227	2500x 1800	26.75	100	0	0	0	0	4.99	18.7	12.14	45.4
O	Gravity flow	13.69	1,499	988	500x 400	6.72	49.1	6.97	50.9	0	0	1.69	12.3	7.30	53.3
P	Gravity flow	106.4		24,568	6500x 2100							7.0	6.6	85.0	79.9

Fig. 3 Population Density Vs. Peak Dwf Rate



is held back by infiltration, depression storage, interception, and so on.

As regards urban areas, rainfall is considered to be mostly held back in pervious areas where infiltration and depression storage are prevalent. The following formula is applicable to urban areas.

$$\frac{\text{Total runoff}}{\text{Total rainfall}} \doteq \text{Ratio of impervious area}$$

This formula shows that an increase in the ratio of impervious area tends to increase the total stormwater runoff and hence to increase the wet-weather overflow. Thus, urbanization exacerbates this problem.

Tables 3 and 4 summarize water quality data obtained from the experimental districts with both combined and separate sewer systems.

To put it simply in terms of average water quality, the overflows from the combined sewer system can be blamed for its failure to meet effluent standards only in suspended solids. The storm runoff from districts with separate sewer systems is not better in terms of suspended solids than the wet-weather overflows from combined sewer systems. Raw sewage, wet-weather combined sewage and storm runoff, are qualitatively compared in Fig. 5.

The flow of raw sewage, stormwater and their loads (BOD_5) in a typical combined sewer system in Japan are schematically represented in Fig. 6.

The existing combined sewer system reduces a generated BOD loading of 4,890 kg/ha/yr (= 4,700 kg/ha/yr + 190 kg/ha/yr) to 1,440 kg/ha/yr; namely, the overall system has a BOD removal efficiency of about 70%.

Table 5 shows the mean values of the concentrations of heavy metals. The heavy metal concentrations in stormwater runoff differ greatly from district to district. In District M, instantaneous concentrations were observed to be 1.80 mg/l for zinc, 1.93 mg/l for copper and 0.76 mg/l for lead, showing that stormwater runoff is not so innocuous. In residential areas, the stormwater runoff has comparatively low BOD, COD and heavy metals concentrations, but depending on localities high concentrations of SS and n-hexane extracts.

Fig. 4a

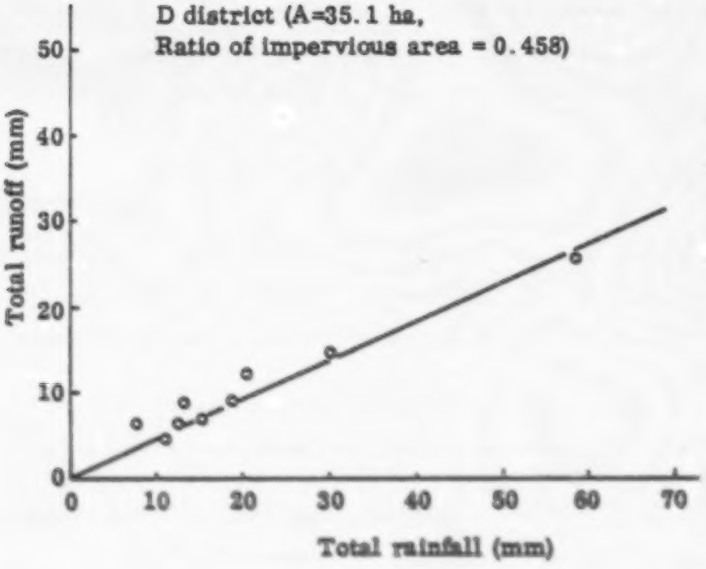

D district (A=35.1 ha,
Ratio of impervious area = 0.458)
Total runoff (mm)
0
10
20
30
40
50
Total rainfall (mm)
0
10
20
30
40
50
60
70

Fig. 4b

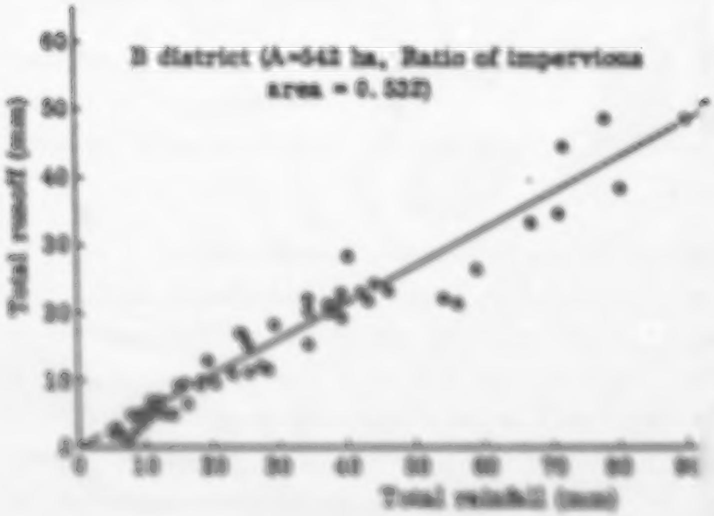

B district (A=642 ha, Ratio of impervious area = 0.532)
Total runoff (mm)
0
10
20
30
40
50
60
Total rainfall (mm)
0
10
20
30
40
50
60
70
80
90

Fig. 4c

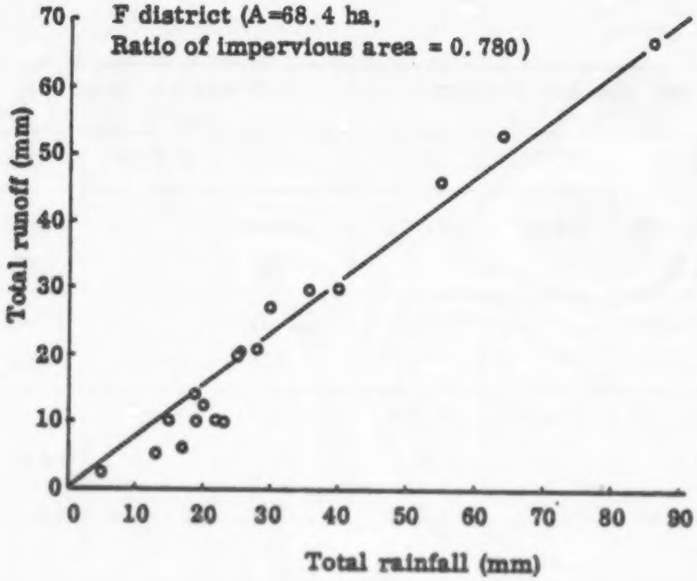


Fig. 5 Typical Quality of Raw Sewage, Combined Sewage and Storm Runoff

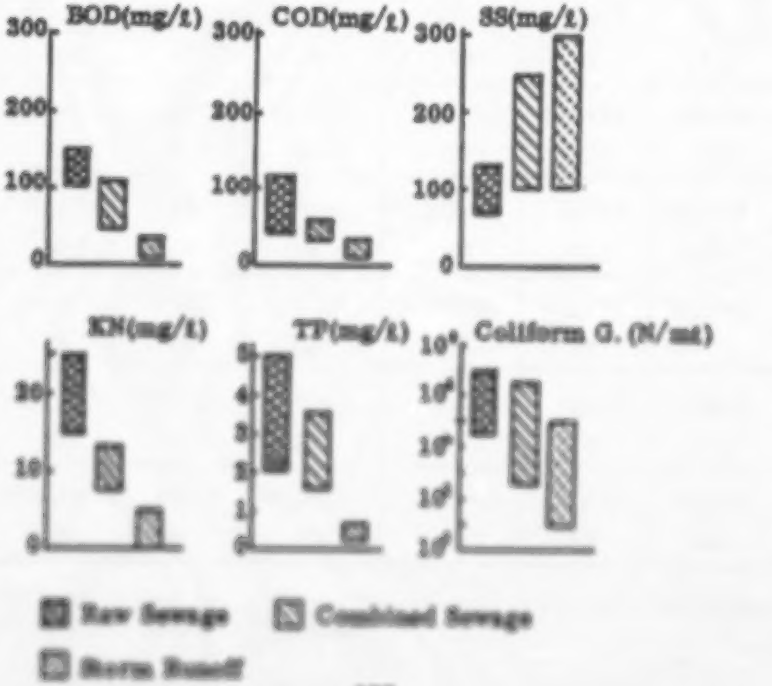


Table 3. Dry Vs. Wet Weather Quality of Combined Sewage

	Dry Weather Quality (mg/ℓ)			Wet Weather Quality (mg/ℓ)			Number of Rainfalls
	BOD_5	COD(Mn)	SS	BOD_5	COD(Mn)	SS	
A	21∿560 138	6∿158 42	14∿163 70	10∿465 83	18∿146 49	44∿604 197	9
B	23∿151 103		12∿157 83	11∿613 82		55∿716 160	11
C	12∿229 127	10∿136 53	12∿205 88	7∿347 76	9∿304 55	10∿1250 134	9
D	35∿281 110	24∿191 49	19∿258 76	13∿182 59	24∿126 31	49∿240 117	10
E	16∿374 123	15∿293 116	8∿844 151	13∿503 46	9∿268 51	4∿1090 249	13
F	39∿180 131	12∿67 52	26∿184 74	7∿247 39	4∿46 22	8∿565 80	12
G	8∿205 127	9∿63 55	13∿125 89	34∿288 81	21∿153 50	52∿550 224	10
H	66∿691 303	35∿359 211	22∿194 119	52∿522 194	19∿265 137	14∿668 192	9
I	11∿180 95	16∿76 40	19∿202 84	3∿260 68	6∿81 43	13∿368 148	10
J	14∿203 108	17∿148 65	12∿187 80	10∿314 39	13∿167 44	25∿294 101	10
K	6∿414 111	8∿129 33	7∿328 67	26∿840 109	13∿280 33	21∿2130 324	9
L	21∿297 144	14∿130 33	18∿340 113	10∿1791 114	8∿198 39	20∿2830 202	10

*upper:range of quality, lower:weighed mean quality

Table 4. Quality of Storm Water Runoff

	Quality (mg/ℓ)					Number of Rainfalls
	BOD	COD(Mn)	SS	TP	KN	
M	1.7~980 40.9	2.9~190 27.2	9.4~2561 267.3	0.05~2.96 0.45	1.40~71.50 19.28	10
N	1.2~88.6 10.5	5.1~72.8 24.2	3.2~490 309.9	0.003~1.44 0.24	0.61~5.73 2.70	10
O	3.1~70.4 28.8	0.4~13.8 4.9	8.3~279 107.8	0.06~1.67 0.24	(TN) 0.0~11.2 0.63	4
P	3.1~14.3 9.3		18.0~476 149.9			7

*upper:range of quality, lower:weighed mean quality

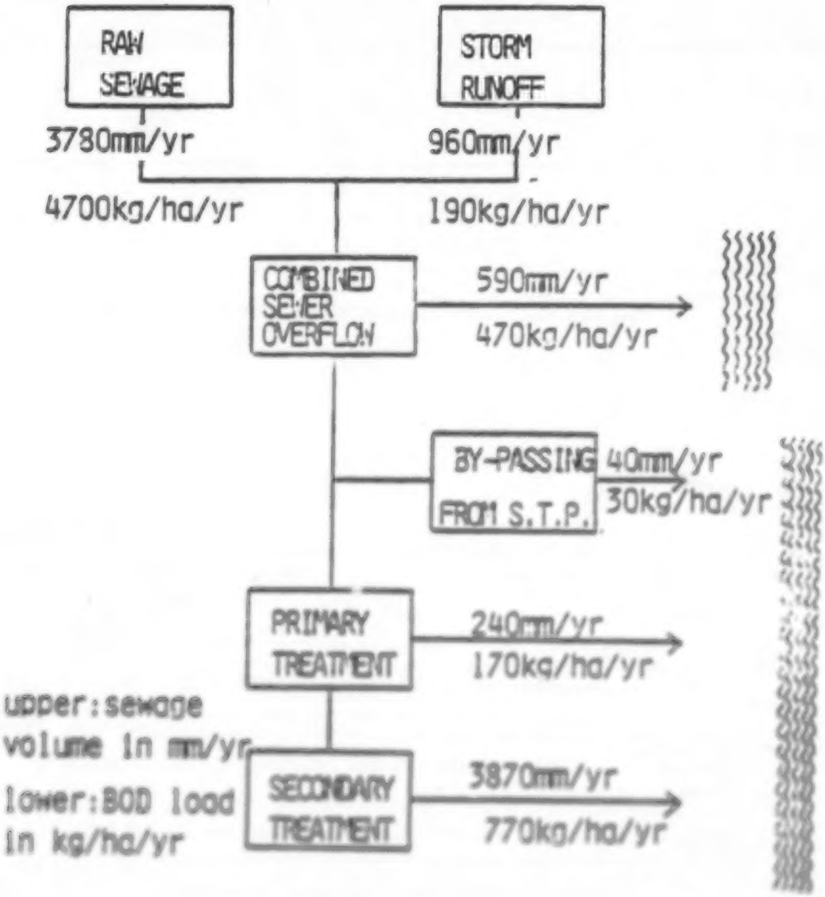
Table 5. Concentration of Heavy Metals

(mg/l)

	Zn	Cu	Pb	Cd	Ni	Cr
Raw Sewage	0.030 ~2.230 (0.473)	0.013 ~0.844 (0.041)	0.000 ~0.040 (0.003)	0.000 ~0.002 (0.000)	0.000 ~0.002 (0.000)	0.000 ~0.001 (0.000)
Wet Weather Combined Sewage	(0.412)	(0.080)	(0.030)	(0.001)	(0.012)	(0.013)
Storm Water Runoff	(0.390)	(0.087)	(0.093)	(0.001)	(0.006)	(0.000)

( ): average concentration, figures of wet weather combined sewage are the average of 12 districts, figures of storm water runoff are the average of 3 districts

Fig. 6 Typical Combined Sewerage in Japan


RAW SEWAGE
STORM RUNOFF
3780mm/yr
4700kg/ha/yr
960mm/yr
190kg/ha/yr
COMBINED SEWER OVERFLOW
590mm/yr
470kg/ha/yr
BY-PASSING FROM S.T.P.
40mm/yr
30kg/ha/yr
PRIMARY TREATMENT
240mm/yr
170kg/ha/yr
SECONDARY TREATMENT
3870mm/yr
770kg/ha/yr
upper: sewage volume in mm/yr
lower: BOD load in kg/ha/yr

2.3 Mathematical models

As one way of attacking combined sewer problems, simulations by mathematical models are used with which we can assess the effect of presumed solution on the combined sewer system.

The Public Works Research Institute of Ministry of Construction has developed three mathematical models of different levels to handle several situations.

The loading rate of wet-weather combined sewage is closely related to flow rate. (Figs. 7a, 7b)

In the case of SS loading, the correlation with flow rate is particularly significant.

For BOD, COD and SS, the following formulas have been established.

$L = C_1 \cdot (Q - Q_c)$ (for BOD, COD) ... (1)

$L = C_2 \cdot Q(Q - Q_c)$ (for SS) ... (2)

Where, L is the loading rate (g/sec.); Q is the flow rate (m^3/sec.); Qc is the critical flow rate, (m^3/sec.); and C is a constant.

Qc is not the critical shear stress in a strict sense; in Figs. 6, 7a, 7b it is found that its value approaches Qc (which is larger than sero) asymptotically.

Thus, it is logical to use the critical flow rate Qc in Formulas (1) and (2).

According to formulas (1) and (2), the loading rate is a single-valued function of flow rate; but actually, it is not so simple. The flow rate versus loadings is shown in Figs 8a, 8b, 8c, 9a, 9b and 9c, in which plots are numbered in the order of sampling.

It is found that BOD and SS load plots form a clockwise loop. It is reasoned that this looping is caused because wet weather loading is dependent at once on the flow rate and residual loads and also because the residual loads will lower toward the end of a rainfall period.

Fig. 7a Relation Between Flow Rate and BOD Loading

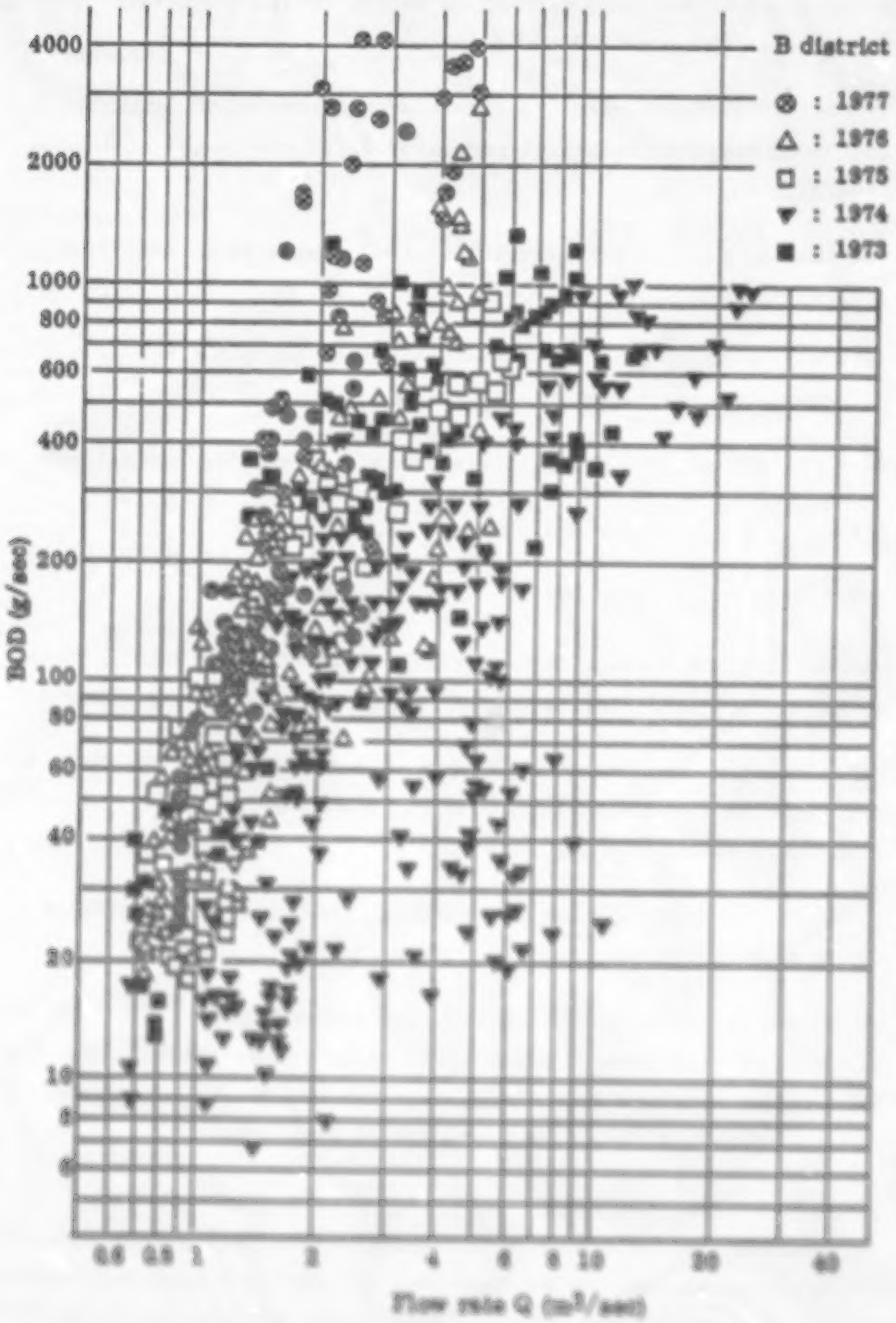


Fig. 7b Relation Between Flow Rate and COD Loading

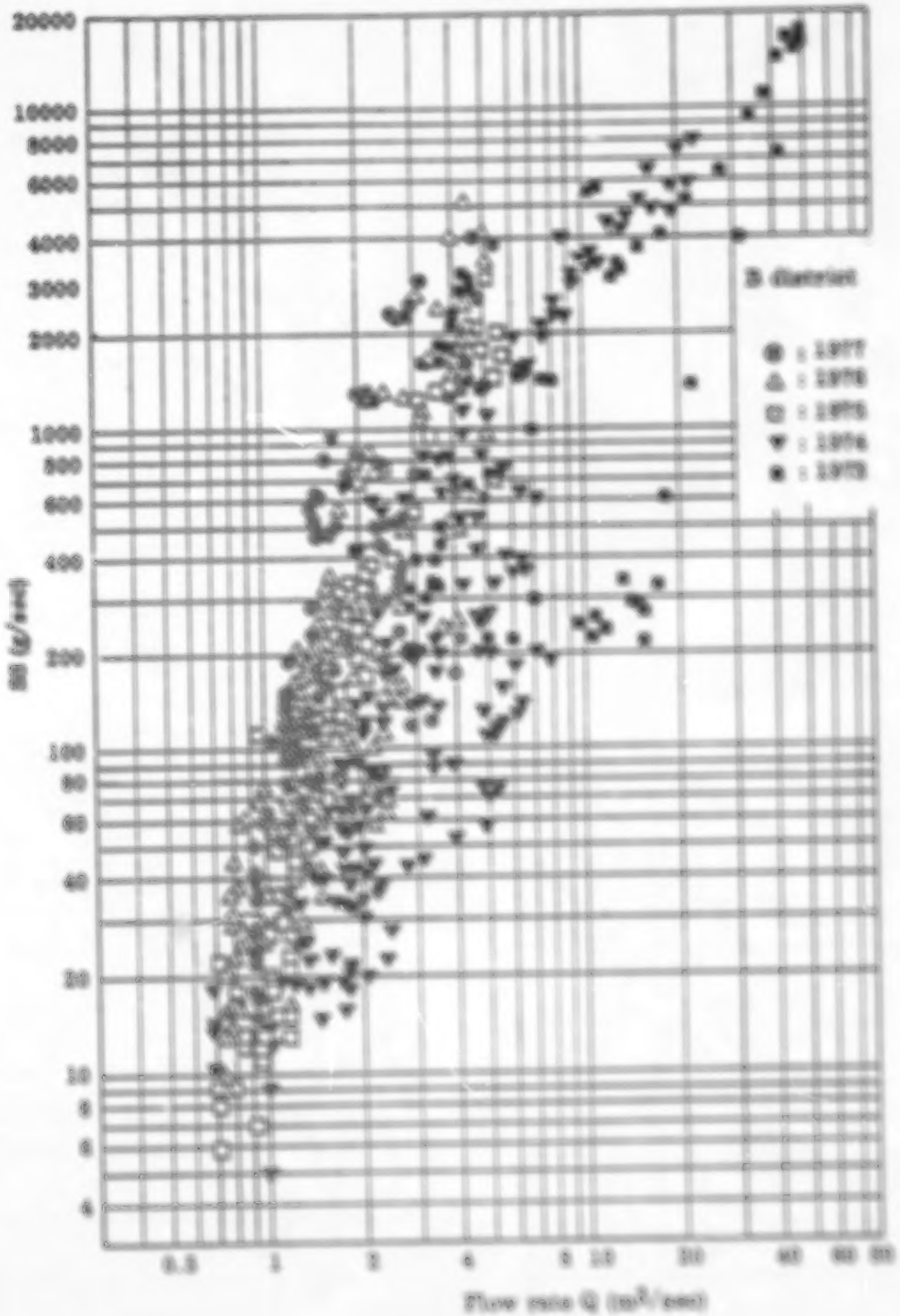


Fig. 8a

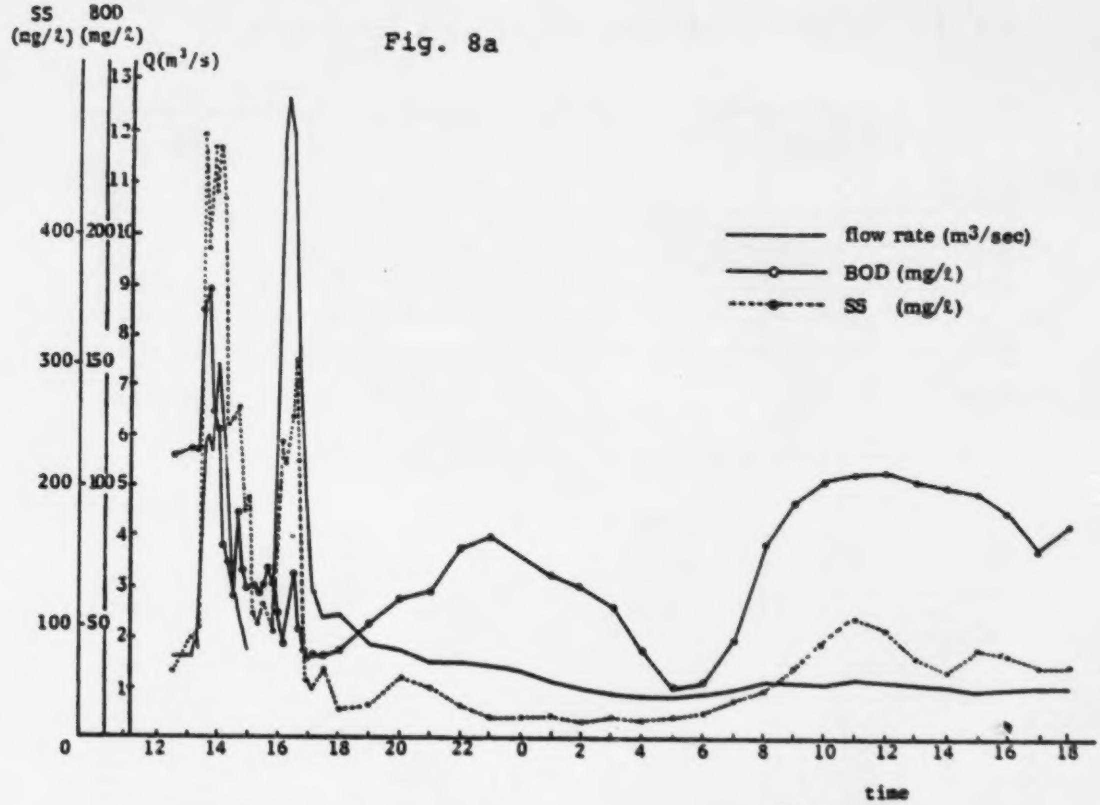
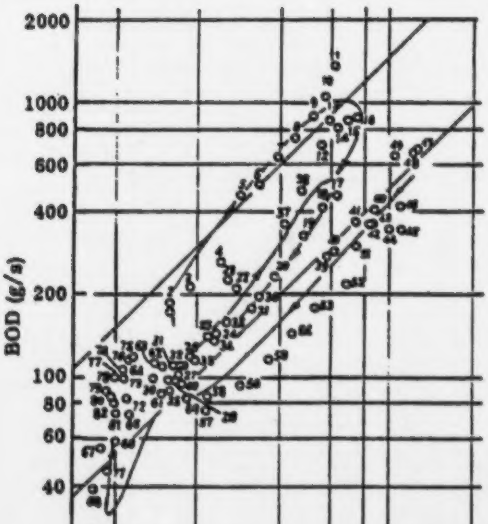


Fig. 8c

Fig. 8b



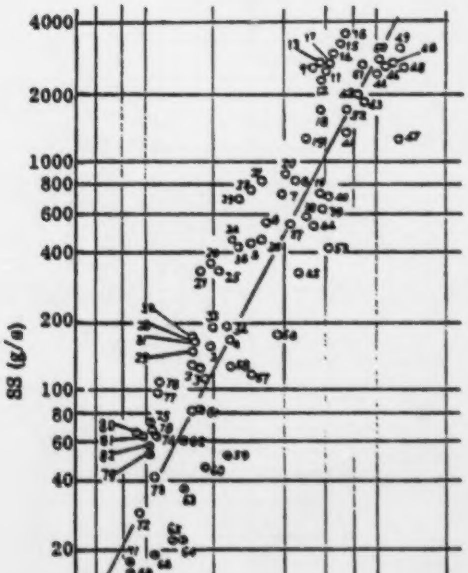


Fig. 9a

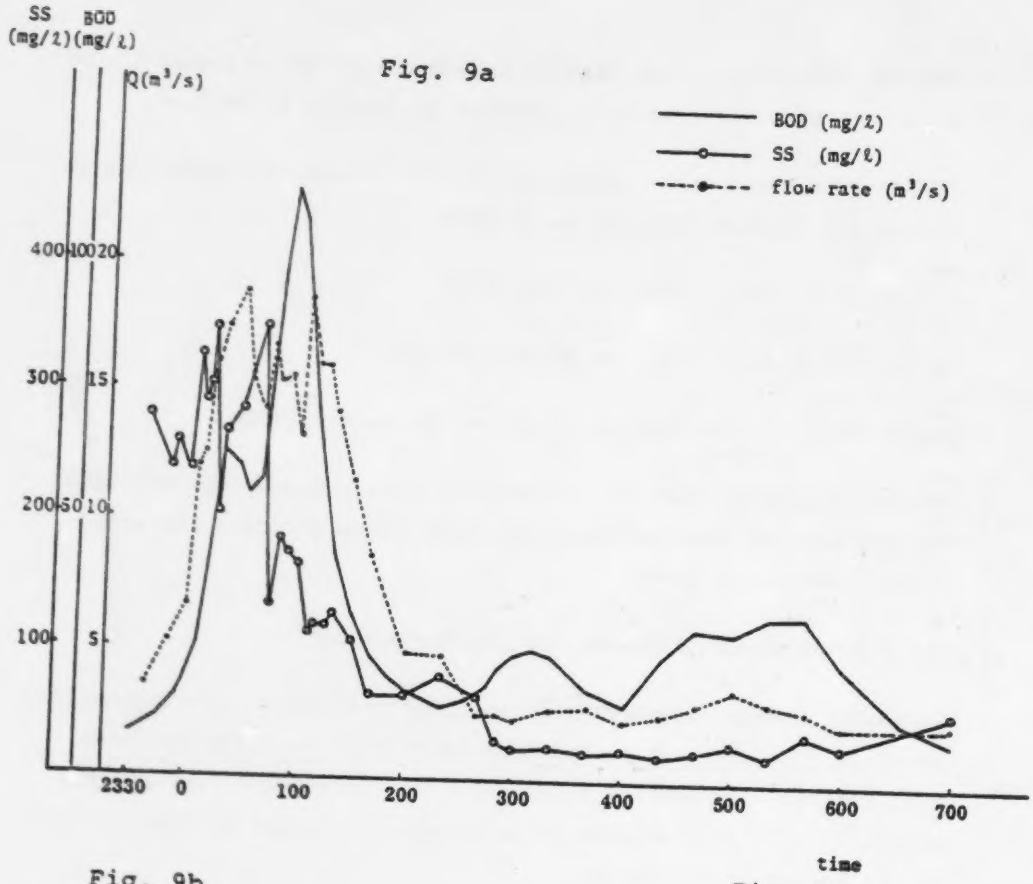


Fig. 9b

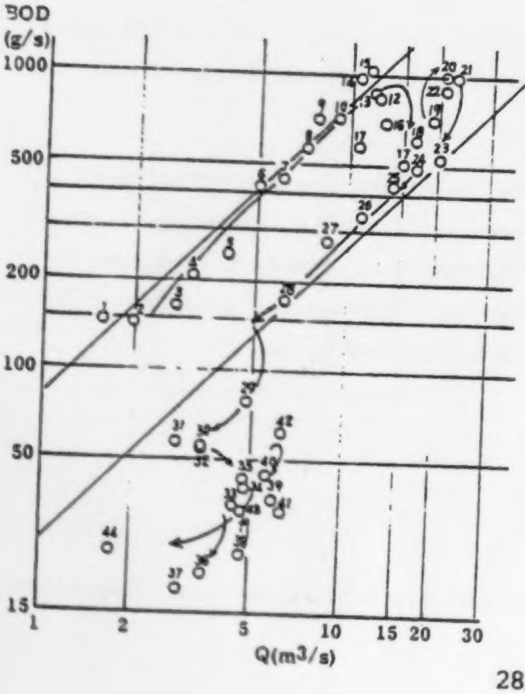
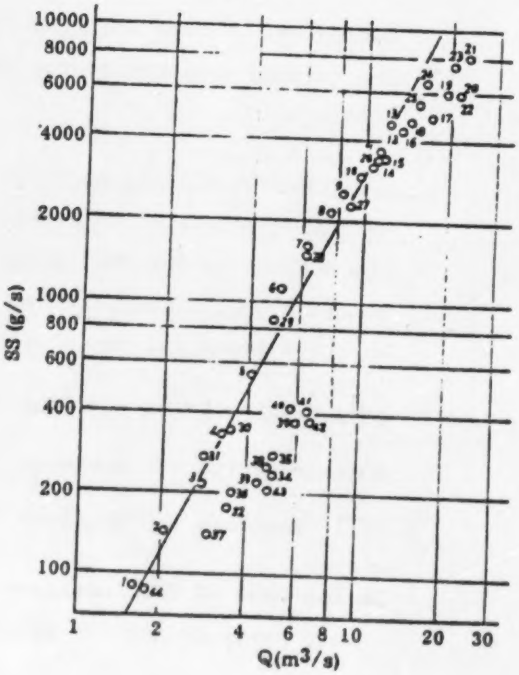


Fig. 9c



BOD and COD loadings are largely influenced by the residual loads, while the effect of residual loads on SS loading is smaller.

Taking into account the effect of residual loads, the equations of motion for loading rate are as follows:

$L = C_1 \cdot P^2(Q - Qc)$ (for BOD and COD) ... (3)

$L = C_2 \cdot P \cdot Q(Q - Qc)$ (for suspended solids) ... (4)

Where, P is the residual load (g) on the sewer bottom.

Fig. 10 compares loads for wet-weather combined sewerage and loads for dry-weather combined sewerage, both being observed for about eight consecutive days.

The hatched sections denote wet weather periods.

Generally speaking, wet-weather loadings are higher than dry-weather loadings, but dry-weather loadings just after the rain are lower than ordinary dry-weather loadings. This is because just after a rainfall, part of the load in raw sewage is trapped or precipitates in the bottom of the sewer.

During wet weather when the velocity is high, these accumulated loads are carried away increasing wet-weather loadings above dry-weather loadings.

For BOD, COD and SS, positive loads are defined as the sum of wet-weather loadings which exceed dry-weather loadings. Negative loads are defined as the sum of loading differences between dry-weather loadings just after rain and the ordinary dry-weather loadings. Positive loads and negative loads are compared below.

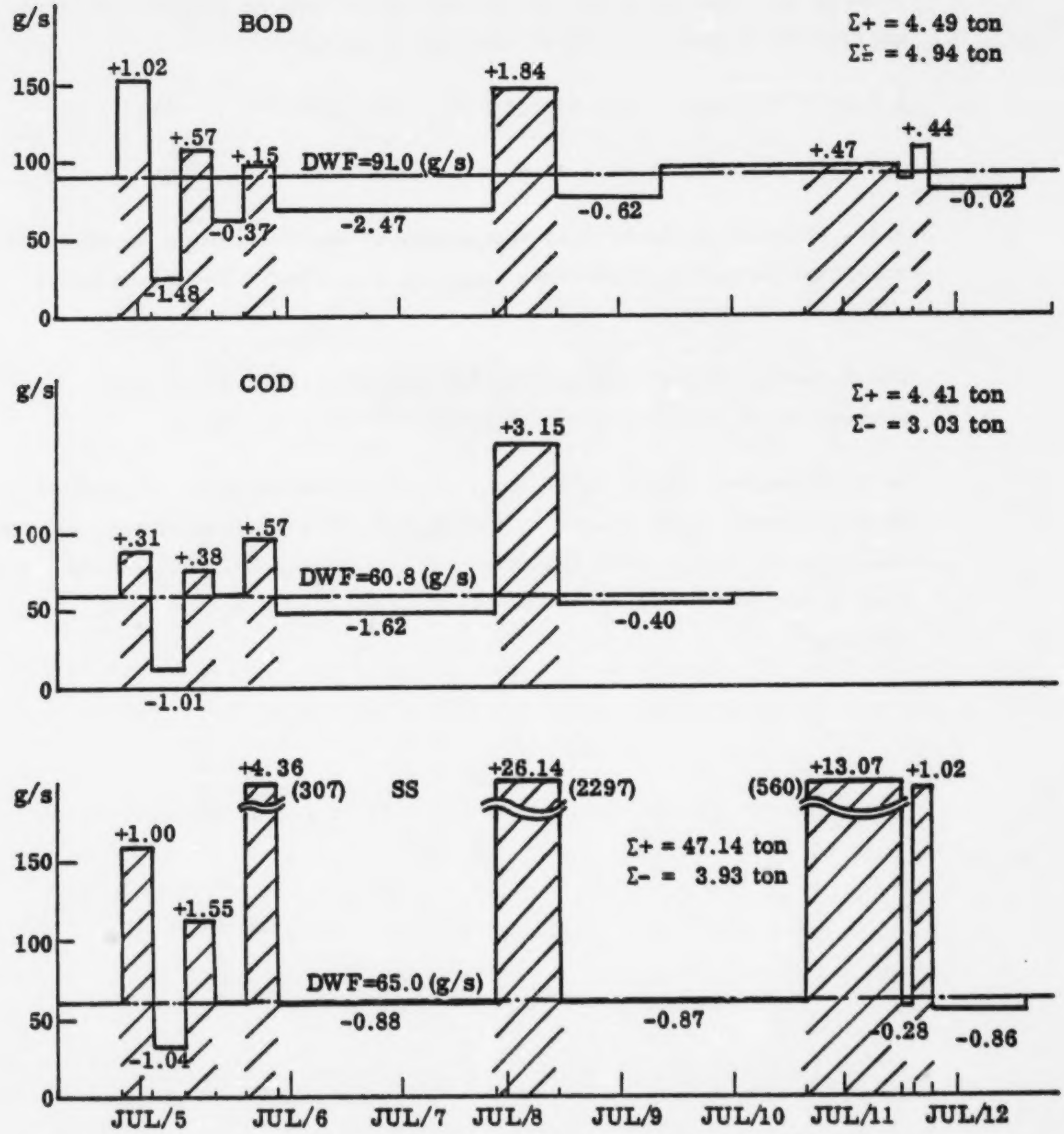
BOD: Positive + = Negative -

COD: Positive + > Negative -

SS : Positive + >> Negative -

In the case of BOD, positive loads are equal to sewage loads deposited in the sewer during dry weather.

Fig. 10 Relation Between Positive Loads and Negative Loads



In the case of COD and SS, positive loads are larger than negative loads. That is, part of the positive loads is contributed by inflow loads from roads surfaces, roofs and sediments in catch basins, together with stormwater.

Accordingly, the equations of motion for COD and SS can be rewritten as follows to take account of external loadings.

$$L = C_1 \cdot P^2 (Q - Qc) + (1/3.6) \cdot k_1 \cdot S \cdot r \quad \text{(for COD)} \quad \ldots (5)$$

$$L = C_2 \cdot P \cdot Q (Q - Qc) + (1/3.6) \cdot k_2 \cdot S \cdot r \quad \text{(for SS)} \quad \ldots (6)$$

Where, k is a constant; S is the amount of residual loads on road surfaces, in gutters, catchment basins, etc.; and r is the rainfall intensity (mm/hr.).

The formulas for projecting BOD, COD and SS loading rates are summarized in Table 6, together with continuity equations.

If a stormwater runoff model (e.g., modified RRL method) is combined with the above loads forecasting formulas, we can evaluate wet weather loads when rainfall data are given. Figs. 11a, 11b and 11c demonstrate that the observed values and the calculated values are in good agreement.

Table 6. Equations of motion and continuity for predicting wet weather combined sewer loadings

BOD

$$L = C_1 \cdot P^2 \cdot (Q - Qc)$$

$$\frac{dP}{dt} = \text{dwf loading} - L$$

COD

$$L = C_2 \cdot P^2 \cdot (Q - Qc) + \frac{1}{3.6} k_1 \cdot S \cdot r$$

$$\frac{dP}{dt} = \text{dwf loading} - C_2 \cdot P^2 \cdot (Q - Qc)$$

$$\frac{dS}{dt} = a_1 - \frac{1}{3.6} k_1 \cdot S \cdot r$$

SS

$$L = C_3 \cdot P \cdot Q \cdot (Q - Qc) + \frac{1}{3.6} k_2 \cdot S \cdot r$$

$$\frac{dP}{dt} = \text{dwf loading} - C_3 \cdot P \cdot Q \cdot (Q - Qc)$$

$$\frac{dS}{dt} = a_2 - \frac{1}{3.6} k_2 \cdot S \cdot r$$

L = loading rate (g/sec)
C = coefficient
P = residual load in the sewer line(g)
Q = flow rate (m^3/sec)
Qc = critical flow rate (m^3/sec)
k = coefficient (1/mm)
S = residual load on road surface, roof-top, catch basin, gutter, and so on (kg)
r = rainfall intensity (mm/hr)
a = loading supply rate (g/sec)

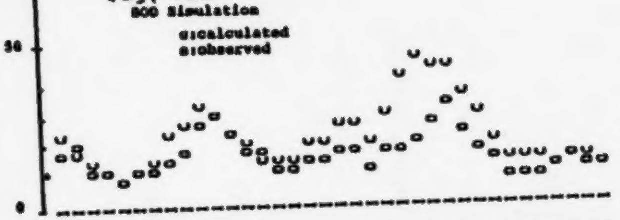
k(COD) = 0.10 (rough surface street)
--- 0.15 (smooth surface street) (1/mm)

k(SS) = 0.20 (rough surface street)
--- 0.30 (smooth surface street) (1/mm)

a(COD) = 1.0 (inactive area)
--- 5.0 (active area) (kg/ha/day)

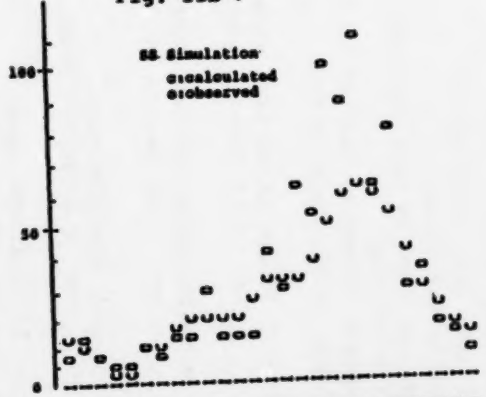
a(SS) = 10 (inactive area)
--- 50 (active area) (kg/ha/day)

Fig. 11a



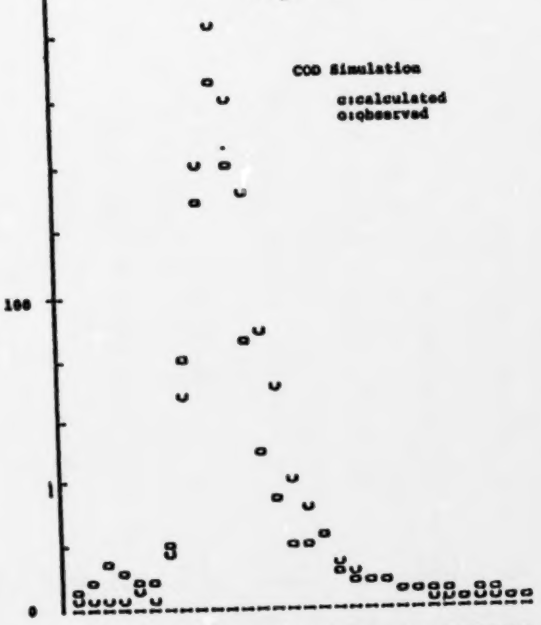
observed (6/5)	calculated (6/5)
18.144	23.840
21.900	16.524
12.832	13.639
10.700	10.402
9.216	8.742
10.961	11.449
11.392	13.842
13.184	22.217
16.401	26.662
27.170	31.204
30.940	29.131
24.600	23.824
18.200	20.509
17.520	15.534
11.748	13.700
10.425	14.904
13.360	19.449
13.832	21.886
18.531	26.730
16.827	26.697
12.922	21.967
17.459	28.939
18.467	41.716
19.062	46.502
27.181	44.396
32.032	43.920
24.414	35.509
17.810	29.649
13.632	21.581
8.517	15.772
8.856	13.853
8.664	13.842
10.981	12.259
13.832	14.128
11.680	13.568
10.472	10.355

Fig. 11b .



observed (6/5)	calculated (6/5)
8.96	14.25
15.87	12.79
8.21	7.01
6.64	2.28
5.57	3.66
12.67	12.64
8.38	10.85
14.73	16.77
14.98	20.37
38.69	19.99
14.88	19.33
13.76	21.26
15.66	26.41
40.99	32.50
38.88	32.57
63.83	32.94
52.84	38.71
101.92	50.14
90.27	60.83
110.38	61.36
62.57	58.99
80.38	54.53
29.61	41.53
35.24	30.47
18.61	23.49
14.10	17.52
7.18	14.50

Fig. 11c



observed (6/5)	calculated (6/5)
6.68	2.30
7.64	3.57
13.90	3.58
11.63	3.90
9.78	4.50
8.72	3.91
21.38	16.94
81.20	68.02
131.82	144.69
169.85	188.19
142.98	164.79
85.52	134.41
49.54	90.59
34.12	71.86
21.84	40.21
19.46	31.45
22.42	24.59
18.15	15.69
9.09	10.40
9.54	8.45
8.33	7.08
6.35	5.43
4.77	4.23
4.59	3.93
4.23	3.59
3.58	3.58
6.13	3.58
4.66	2.53
3.17	1.18
3.57	2.22

3. Measures taken against combined sewer problems

3.1 Improvement of intercepting systems

The improvement of intercepting systems is one of the commonest approaches to the solution of combined sewer overflow problems.

Usually, interceptors are improved in two ways; in one, the height of the overflow weir is increased; in the other, the interceptor is increased in diameter or bypass interceptors are constructed. In Japan, inceptor capacity is usually designed to be about three times the dry-weather peak flow.

But, some places have experienced dry-weather overflow problems when stormwater to be intercepted lowered because of increases in sewage flow. To attack this kind of problem with symptomatic remedies, the improvement of intercepting system has a quick effect.

While this can reduce overflows, it allows the primary effluent to increase. Accordingly, the effect of the intercepter on the entire combined sewer system in reducing pollutant loadings is small. Figs. 12a and 12b shown an example of wet-weather primary effluent. In the sewage treatment plant O, sludge is treated at a separate plant, and there is no return of supernatant from the sludge treatment plant. On the other hand, treatment plant S receives supernatant from the sludge treatment facility into grit chambers. As a result, the quality of the primary settling tank influent is higher than that of raw sewage, and the quality of primary effluent is higher than that of raw sewage which enters the plant during rainfall. In cases like treatment plant S, effluent loads will be increased all the more by increasing intercepting capacity.

In the case of treatment plant O, the wet-weather primary treatment efficiency $(1 - \frac{\text{primary effluent quality}}{\text{plant influent quality}})$ is usually 30% for BOD, 30% for COD and 40% for SS. If the intercepting system is improved and the primary treatment efficiency left unchanged, there will be little reduction of effluent loads from the entire combined sewer system.

Figs. 13a and 13b show a case of increased intercepting capacity in

Fig. 12a

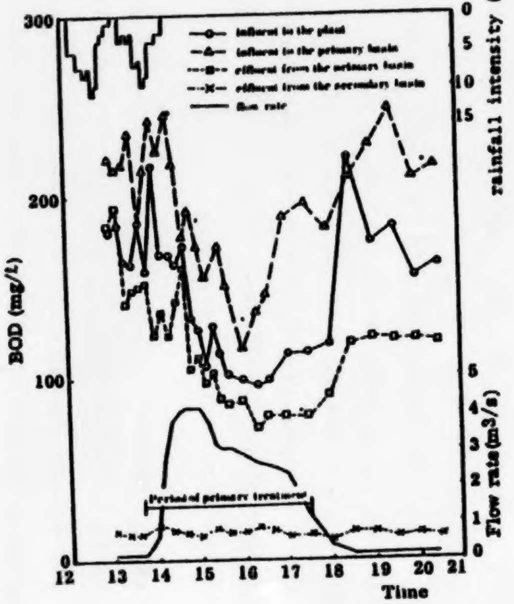
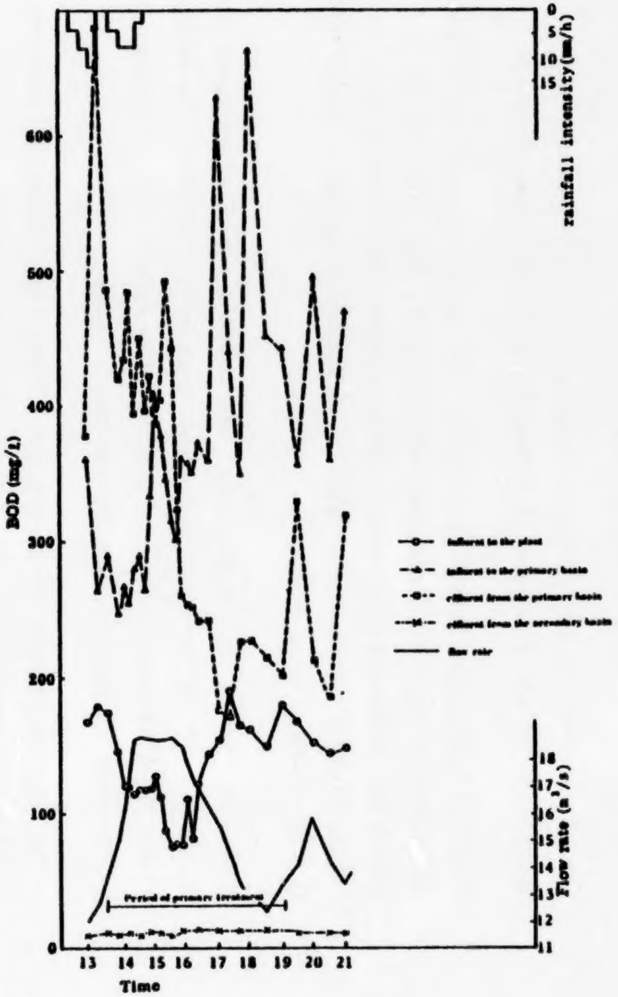


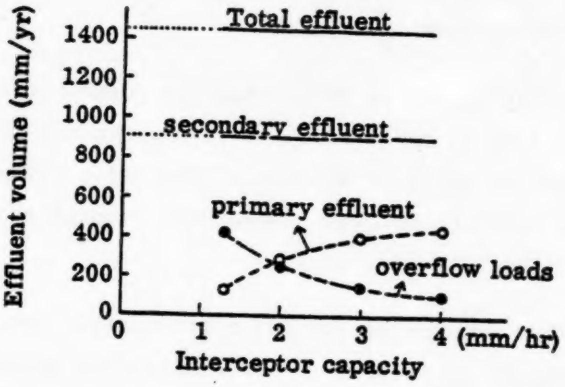
Fig. 12b



which year-round wet-weather effluent flows and loadings are given with respect to the primary treatment facility, the secondary treatment facility and the overall combined sewer system.

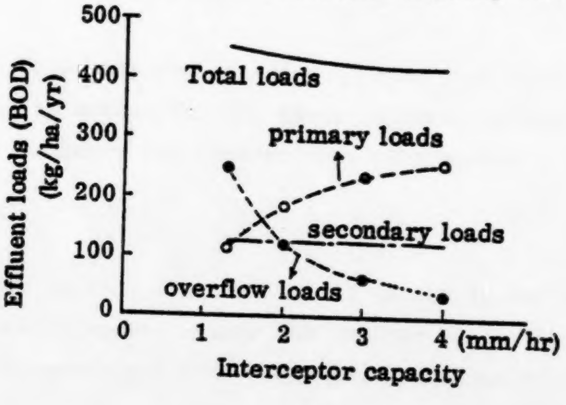
Depending on the conditions of receiving waters, the reduction of overflow loads from diversion chambers in the upper reaches of a river may lead to an improvement in water quality. Usually, however, the improvement of intercepting systems alone cannot lead to improvement of the combined sewer system as a whole unless the primary treatment efficiency or wet-weather treatment system is concurrently improved.

Fig. 13 (a) Effluent volume during wet weather vs. interceptor capacity



ig. 13 (b) Effluent loads (BOD) during wet weather vs. interceptor capacity

* present interceptor capacity =1.3 mm/hr



3.2 Storage

Storage is a truly effective measure in coping with the problems of combined sewer systems.

There are two main storage methods; on-site storage in which stormwater runoff is pooled before being run into the sewerage, and off-site storage in which stormwater runoff is accepted into the sewerage and then pooled in a storage basin connected to the sewerage system.

On-site storage

Porous pavements are beneficial not only to pedestrians, but to stormwater control. As regards the clogging of porous pavement, the Construction Department of the Tokyo Metropolitan Government conducted a survey as shown in Fig. 14.

According to the survey, it is found that the permeability is likely to become around 5 to 10 ml/15 sec., which corresponds to an infiltration capacity of 20 to 40 mm/hr. Even if a pavement loses its permeability, as when it has been loaded with foreign objects, it still seems to offer good infiltration.

Some municipalities require measures for stormwater control as a link in flood control in housing or industrial development.

Fig. 15 is a runoff regulatory basin installed in the lower reaches of a newly developed area.

During fine weather, the runoff regulatory basin is turned into tennis courts.

In some housing complexes, the open spaces between buildings are depressed below the surrounding areas and the inlets of catchment basins are reduced with orifices to control the runoff. (Fig. 16)

Off-site storage

Off-site storage can be mainly classified into in-line storage and off-line storage. In any method, the stored sewage is released to the treatment plant for secondary treatment when the sewage flow has been reduced after rainfall.

Fig. 14 Time Relevant Permeability of Porous Pavement Media

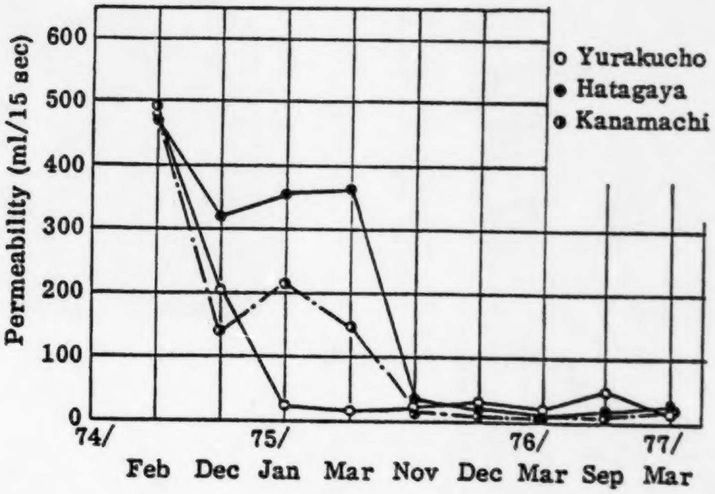
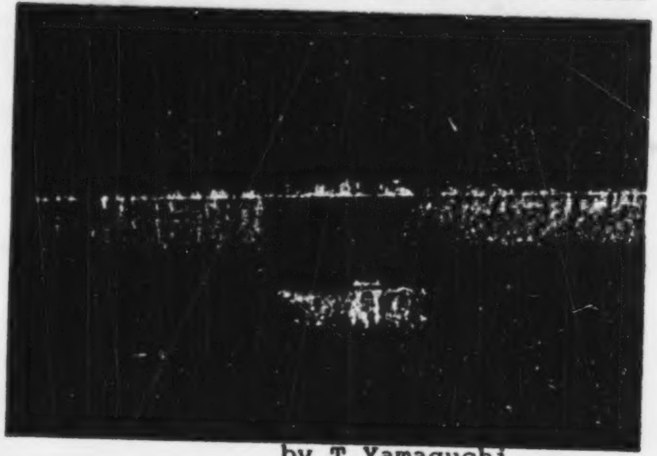


Fig. 15 Runoff Regulatory Basin

by T. Yamaguchi, Practices of Detention Storage in Japan, Public Works Research Institute NO.1174

Fig. 16 Orifice of Catch Basin Inlet



by T.Yamaguchi

For in-line storage, the size of the upper-reach sewer is made larger than that of the lower-reach one in order to control runoff with an orifice. Ichinomiya City uses this method to reduce overflow and at the same time to overcome the chronic problem of inundation in the upper reaches for want of the capacity of the lower sewer system. In Nagoya, the sewers in the middle reach are oversized to provide storage capacity for inundation control. After construction of this storage system, the inundation which the city had experienced about three times a year has not repeated.

Fig. 17 shows the Tennoji-Benten Trunk Sewer under construction by the Osaka Municipal Government. The water level of the river that so far has drained the wet-weather overflows from the combined sewer system is now above the outfalls because of urban development in the upper basin and subsidence in the area, and this trunk sewer is planned to accommodate wet-weather overflows.

A pump station downstream of the trunk sewer will drain overflows into the Yodo, one of the largest rivers in Japan, in the case of heavy rainfall.

For normal rainfalls, overflows will be stored in the trunk sewer. The maximum diameter of the sewer pipe is 6 m, and the aggregate length is about 14 km with a storage capacity of about 88,000 m^3, corresponding to a rainfall of 6.6 mm in covered areas.

Off-line storage is classified into two; one method in which a storage basin is provided around the diversion chamber in the middle part of the sewer system or near the pump station, and another method in which a storage basin is installed at the treatment plant with an oversized intercepter sewer.

The former is better in that it can reduce overflows of the combined sewer system and its loads.

Fig. 18 shows a plan of the detention storage basins installed by the Yokohama Municipal Government near Hodogaya Pump Station.

They total 18, each being 6 m in depth, 2 m in width and 35 m in length. Their total storage capacity is 22,680 m^3, equivalent to

6.4 mm of rainfall. The wet-weather sewage detained in these basins is sent to the treatment plant for secondary treatment when the sewage flow has fallen. By the installation of these basins, the associated combined sewer system has been improved in BOD effluent loadings and is equivalent to a separate sewer system. This improved combined sewer system has been proved to discharge loads 10% less than would be expected from conversion into a separate sewer system.

In a separate sewer system, if the effluent loads due to stormwater runoff were to be reduced, there would be difficulties in the use of existing treatment facilities as stormwater and sanitary sewage have quite different characteristics. It would be almost imperative to install a new treatment plant for processing stormwater runoff. In the case of a combined sewer system, however, wet weather sewage is trained to give characteristics that permit the use of existing facilities. Yokohama's improved combined sewer system has far higher in load removal efficiency than any other separate sewer system without stormwater control facilities.

The combination of storage and treatment goes a long way toward the reduction of pollutant loads and at the same time serves to some degree as a flood control function for receiving waters.

Table 7 summarizes the construction costs for basins.

Overflow sedimentation basin

The overflow sedimentation basin functions just the same as the storage facility unless it is topped up. The difference between a storage basin and an overflow sedimentation basin is in whether the basin in question is to accommodate combined sewer overflow or to discharge it directly into the receiving waters after the basin is topped up. (Fig. 19)

If the storage capacity of the basin is sufficient, there would be no quality problem if combined sewer overflow were to be discharged directly into receiving waters when the basin were nearly full. On the other hand, if the basin's capacity were insufficient, the basin could well be used for sedimentation purposes in expectation of load removal.

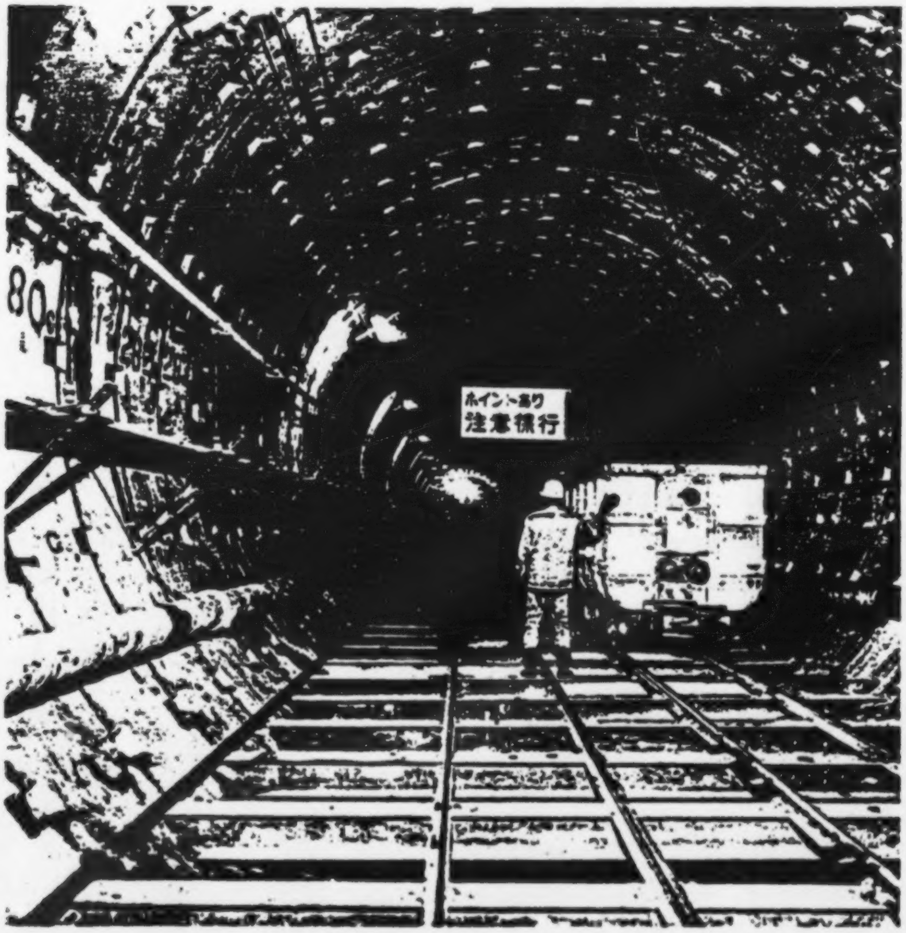
Fig. 20 shows a plan of the overflow sedimentation basins installed at the Nakanoshima Pump Station in Osaka. At present, two basins have been completed.

Figs. 21a, 21b, 21c and 21d show SS removal by these basins. While the basins can clip the peak concentrations of influent, their sedimentary removal rate is not so high. Pollutant removal by the storage of first flush is high, however.

Table 7 Construction Cost of Detention Storage

City	Type	Main Purpose	Storage Capacity (M^3)	Cost (yen/M^3)
YOKOHAMA	off-sewer	poll. control	23,000	
OSAKA	in-sewer	flood control	51,700	338,500
"	off-sewer (sed. basin)	poll. control	630	173,000
NAGOYA	in-sewer	flood control	2,000	38,600
ICHINOMIYA	in-sewer	flood control	2,450	26,500
"	in-sewer	flood control	1,100	13,600
"	off-sewer	poll. control	4,500	39,000
YAMATO-KORIYAMA	off-sewer	poll. control	17,000	16,000

Fig. 17 Tennoji-Benten Trunk Sewer under way


ホイントあり
注意徐行
80

by Osaka Municipal Government

Fig. 18 Detention Storage in Yokohama City

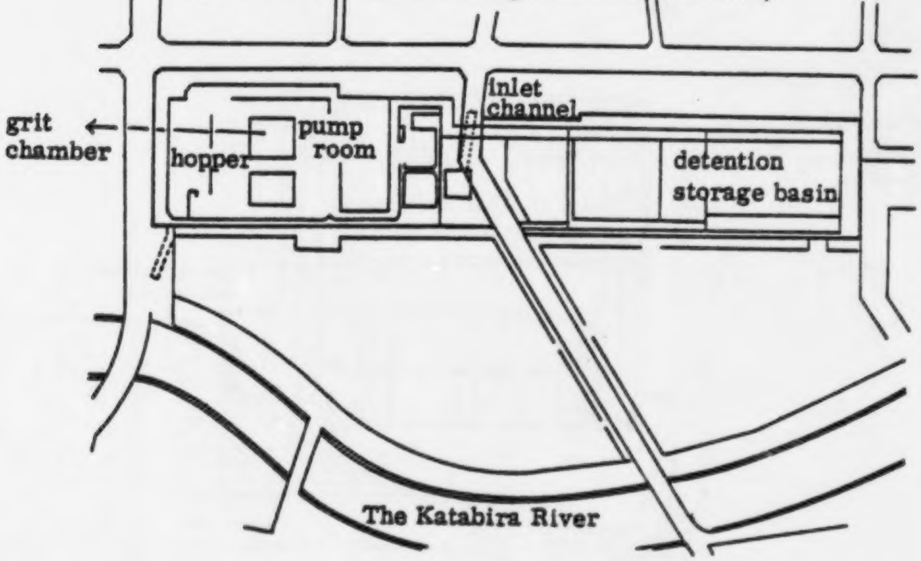

inlet channel
grit chamber
hopper
pump room
detention storage basin
The Katabira River

Fig. 19 System flows of Detention Storage and Sedimentation Basin

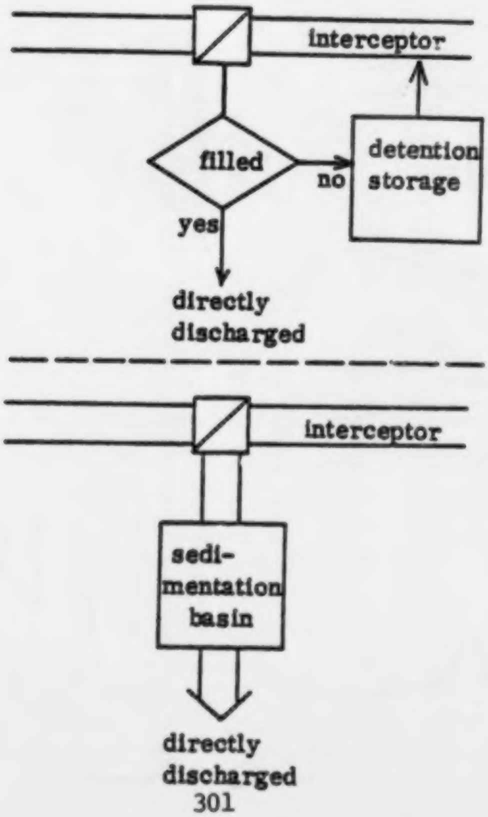

interceptor
filled
no
yes
detention storage
directly discharged
interceptor
sedi-mentation basin
directly discharged

Fig. 20 Sedimentation Basin in Osaka City

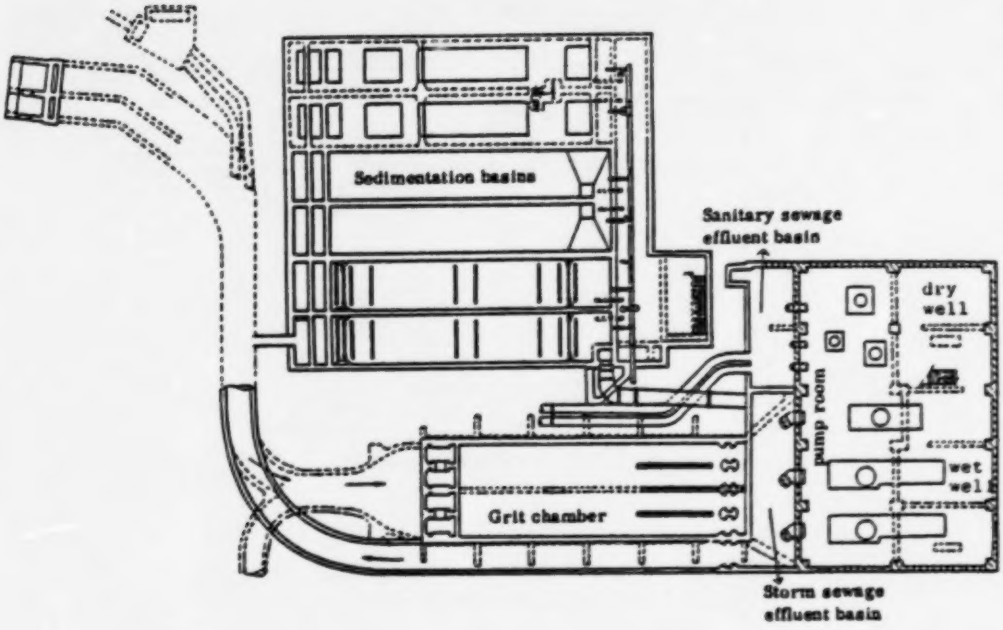

Sedimentation basins
Sanitary sewage effluent basin
dry well
pump room
wet well
Grit chamber
Storm sewage effluent basin

Fig. 21a

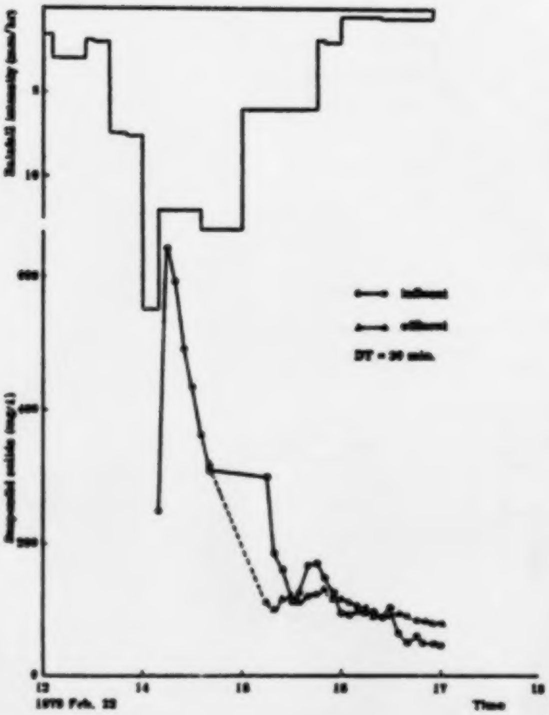


Fig. 21b

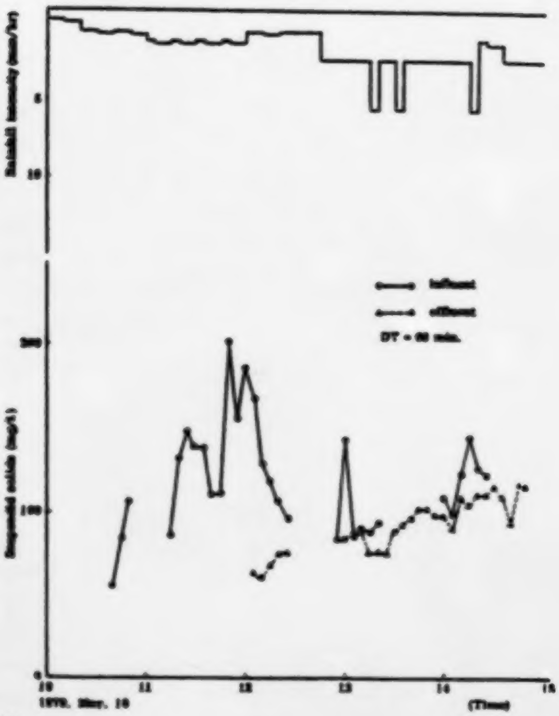


Fig. 21c

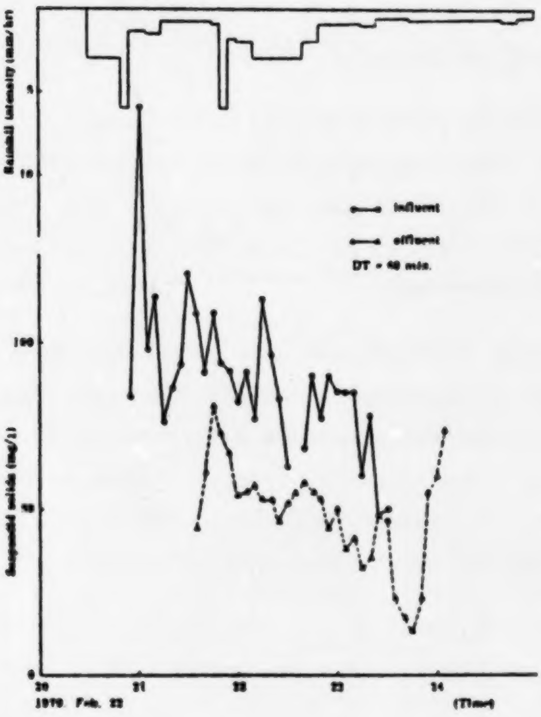
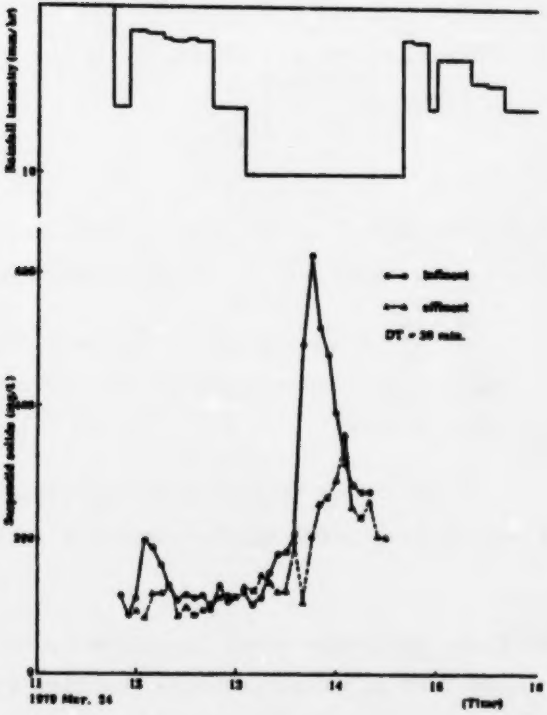


Fig. 21d



3.3 Experimental measures

Swirl regulator/concentrator

Treatment facilities designed solely for combined sewer overflows are extremely disadvantageous from the economic viewpoint. The swirl regulator, however, costs less to construct and needs little or no expenses for maintenance, and is effective as a measure against combined sewer problems.

The Public Works Research Institute of the Ministry of Construction and the Bureau of Sewerage of the Tokyo Metropolitan Government have been experimenting with prototype swirl regulators of 0.9 m and 1.2 m in diameter in order to assess their pollutant removal efficiency. Table 8 shows test results achieved by the Tokyo Metropolitan Government's Sewerage Bureau on artificial turbidity.

Fig. 22 shows a forecast of removal rate of a 12 m ø swirl regulator, obtained by applying Froude's law to Table 8.

The prototype swirl regulator has the drawback that it needs a height difference between the influent sewer and overflow sewer. For this reason, the two organizations are studying the improvement of swirl regulators.

Sewer flushing

It is well known that the flushing effect of stormwater conveys bottom loads off the sewer to cause the so-called first flush of low quality.

A sediment loads survey was conducted in 300 manholes within the B district. Of them, 44 disclosed deposits filling more than a quarter of the invert. (See Fig. 23)

The BOD load in the sewer way estimated by gravimetric analysis and qualitative analysis of several typical deposits to be about 1.0 to 1.5 ton/km^2.

More than 70% of the aggregate sewer length was accounted for by pipes of 500 mm or less in diameter where deposits were formed. Therefore, if sediment in pipes of 500 mm or less can be forced into pipes of larger size, not only the problem of first flush, but a substantial part of the combined sewer problems can be solved.

Water was discharged into an upstream manhole and was sampled from a downstream manhole. (See Fig. 24)

Analysis results for the sampled water are shown in Fig. 25. The pouring rate was 3 ℓ/sec., and the pouring duration was 30 min. To determine the initial loads deposited inside the pipe, water was discharged for 30 min. at a rate of 8 ℓ/sec. after the 3 ℓ/sec. test.

It was found that about 50% of the initial BOD load (326 g/73.8 m of sewer) could be washed away in 15 min. by water discharge at 3 ℓ/sec.

Catch basin

A catch basin is usually constructed to have a sand arresting pit more than 15 cm deep at its bottom. (See Fig. 26)

Debris and sand carried in from road surfaces should be trapped in this pit. According to a survey conducted by the Public Works Research Institute of the Ministry of Construction, about 15% of catch basins were found to be totally filled up; those catch basins where the sediment height exceeded 10 cm accounted for more than a quarter of the total. (See Fig. 27) In view of this, the regular cleaning of catch basins is one solution to overflow problems.

Other experimental projects include road cleaning, which was started in 1979, no definite results are available yet.

Fig. 22 Removal Efficiency of Swirl Regulator Concentration

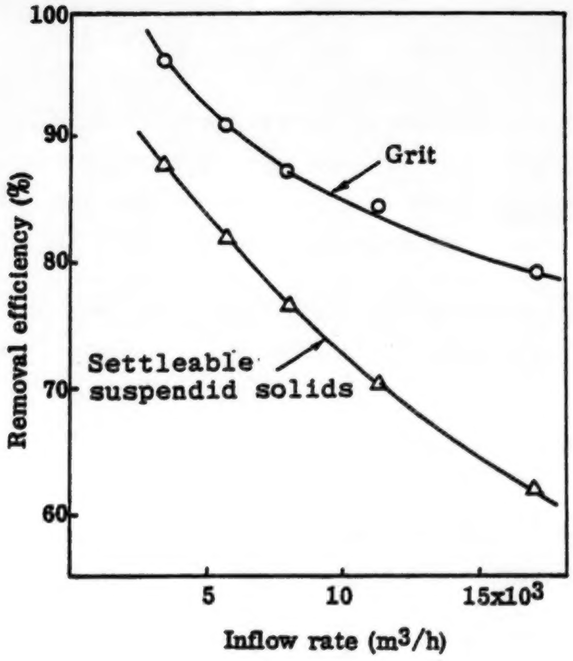


Fig. 24 Experiment of Sever Flushing

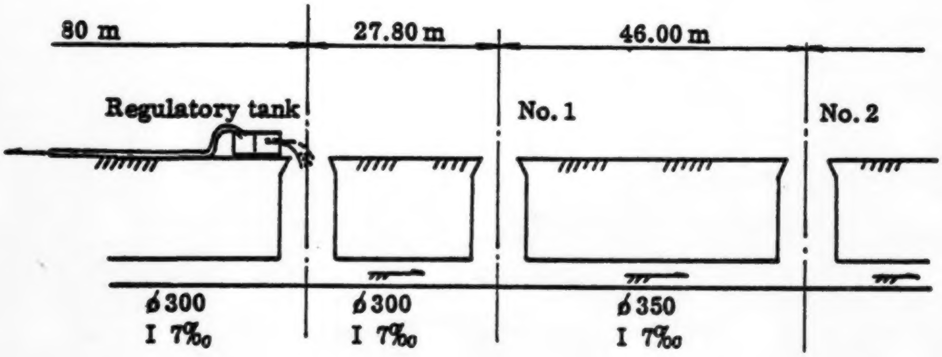

80 m
27.80 m
46.00 m
Regulatory tank
No. 1
No. 2
ϕ300
I 7‰
ϕ300
I 7‰
ϕ350
I 7‰

Fig. 23 Deposits in The Sewer

Fig. 25 Quality Variation During Sever Flushing

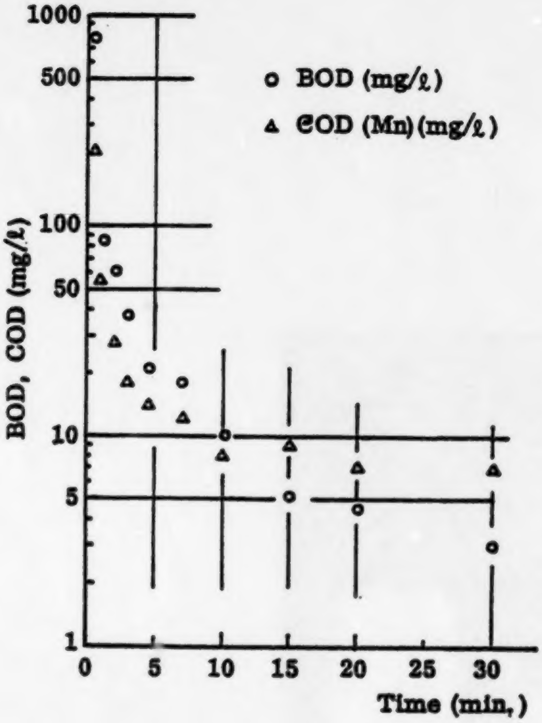
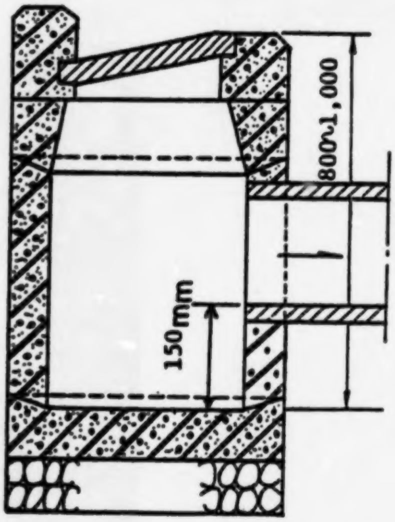


Fig. 26 Catch Basin



800~1,000
150mm

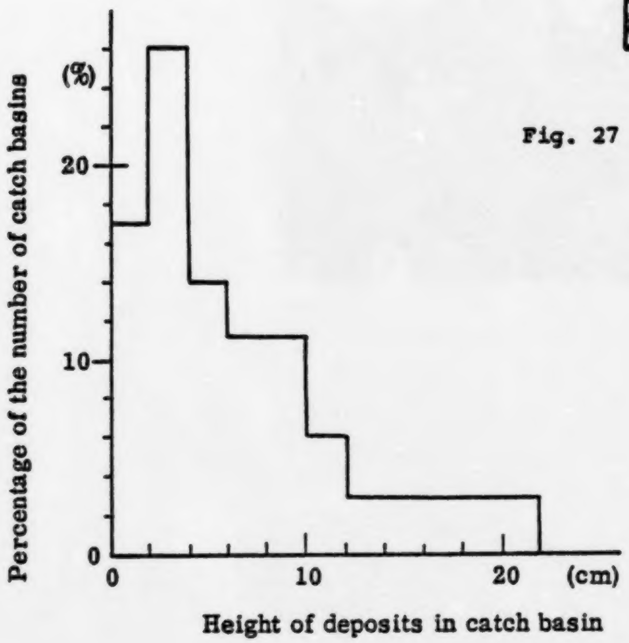


Fig. 27

Table 8. Removal Efficiency of Swirl (D=1.2m)

	Specific gravity	Diameter mm	Moisture content %	Q= 10.8m³/hr	Q= 18m³/hr	Q= 36m³/hr	Q= 54m³/hr	Q= 72m³/hr	Q= 108m³/hr
Zeolite	2.13	0.8~2.4	3.7				99.9		99.9
"	2.13	0.5~1.4	3.7					99.8	99.9
Powder carbon	1.50	0.5~0.6	18.9		99.9	99.8	95.7	94.7	
Anthlasite	1.41	2.0	0.7				99.8	99.2	
"	1.41	1.0	0.7				96.8	97.4	
"	1.41	0.5	0.7				94.7	95.5	
Italian millet	1.23	1.5~1.7	6.5		97.3	93.4	90.3	88.9	
Rice hull	1.09	2.0 3.2~3.5 7.0~7.5	8.9		97.5	92.2	87.4	73.9	
Barnyard grass	1.06	1.8~2.0	9.0		92.9	72.9	55.6	29.5	
Styrol	1.05	3.0	0.1		97.9	90.0	83.0	72.7	
"	1.05	2.0	0.1	99.6	92.0	77.8	70.2		
"	1.05	1.2	0.1	98.3	86.7	66.8	48.1		
"	1.05	0.7	0.1	90.3	72.9	46.2	28.0		

4. Conclusions

Combined sewer problems in Japan and their countermeasures have been discussed.

To tackle combined sewer problems, we should firstly consider how to make the most of existing facilities. The combination of storage and treatment facilities will be appropriate.

Secondly, we should decide the extent that the effluent loads should be reduced.

For this, it is necessary to establish a model forecasting the quantity and quality of combined sewer overflows, as well as a model for evaluating the quality of receiving waters.

At present, it is not clean what impact combined sewer overflows have on the quality of receiving waters.

This is particularly the case with the relationships between overflow loads and the dry-weather quality of receiving waters, which are an important theme awaiting our study efforts.

Every engineer accepts that the simpler a mathematical model is, the better it is. Any drainage area has limits to the geographical and hydrological data that can be collected specific to it.

Thus, the use of black box type models as introduced in this paper is worth trying.

While the separate sewer system has been improved little, it will incur no overflow problem for the time being except for in some water characteristics.

As regards storm runoff from highly congested areas, separate sewer systems will not be sufficient, and additional measures should be provided. In Japan, there are many infiltrated sanitary sewers, and surveys on inflow/infiltration control are being carried out.

Seventh US/JAPAN Conference
on
Sewage Treatment Technology

WATER QUALITY IMPROVEMENT IN YODO RIVER AND SEWAGE WORKS IN KYOTO

May 21, 1980
Tokyo, Japan

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Sewage Works Bureau,
City of Kyoto

WATER QUALITY IMPROVEMENT IN YODO RIVER AND SEWAGE WORKS IN KYOTO

1. Preface 313
2. Outline of Yodo River 313
    2.1 Characteristics of Catchment Area 313
    2.2 Water Utilization 315
    2.3 Outline of Water Quality 317
    2.4 Control by Laws 319
3. Sewage Works Project in Kyoto City 321
    3.1 Project Outline 321
    3.2 Improvement of Water Quality in Yodo River 323
4. Conclusion 329

1. PREFACE

The Yodo River running through the five prefectures of Osaka, Kyoto, Shiga, Nara and Mie has a catchment area of 7,280 km^2 including Osaka and Kyoto cities and other growing satellite cities. With a total population of 9,700,000 in the catchment area, the Yodo River is the foundation for social, economic, cultural and industrial activities in western Japan.

Among the Japanese rivers having large flow fluctuations, the Yodo River is comparatively flow-stabilized river since Lake Biwa is located in the upper stream of the river. The catchment area of the Yodo River has prospered since olden days, depending much upon the stabilized water resources. Today, the Yodo River brings blessings of nature to the area as water resources for water supply, industrial and agricultural uses.

The Yodo River, which has such an important role in the Kinki regional sphere, has been facing some problems because of pollution of water quality, outbreak of red water in Lake Biwa, offensive smells for waterworks, increase of ammonia nitrogen in the lower basin of the river; all affected by an increase of pollutant loads brought by the rapid development of industry and the concentration of population in its catchment area.

The City of Kyoto, situated in the midstream basin of the Yoro River, is a great city of 1,460,000. All wastewaters from the city and its neighboring area are discharged into the Yodo River, but on the other hand, the Hanshin (Osaka-Kobe) area in the lower stream basin of the river is taking water on recycling process system.

As a result, the water pollution of the Yodo River has been emerging as problem from an early stage. In 1958, the Council for Yodo River Water Pollution Control was inaugurated by 21 organizations including the central government and related local governments and water using bodies, and river supervisor, user and discharger have been exerting efforts to prevent pollution of the Yodo River water.

In order to carry out the improvement of water quality in the Yodo River, the construction of sewage works in the catchment area of the river is the most effective way. Especially, the sewage works in Kyoto City is most importance, so Kyoto City has positively expanded sewerage systems as one of the important policies of the city.

With the extension and construction of the sewage works and strengthening of control over wastewaters from factories in Kyoto City showing good effects, water pollution in the Yodo River has been gradually reduced since the peak year of 1969 and the river has been restored to the present environmental standards. Here is a report about the process for improvement.

2. OUTLINE OF YODO RIVER

2.1 CHARACTERISTICS OF CATCHMENT AREA

The catchment area of the Yodo River is divided into the five portions of the Uji River, the Katsura River, the Kizu River, Lake Biwa in the upper stream basin of the Uji River and the main stream of the Yodo River after joining the three rivers. (See Fig. 1 and Table 1).

The hydrogeographic characteristics of the sub-catchment areas contribute to a good water balance for the entire basin of the Yodo River and are providing

Fig. 1 Map of Yodo River Catchment

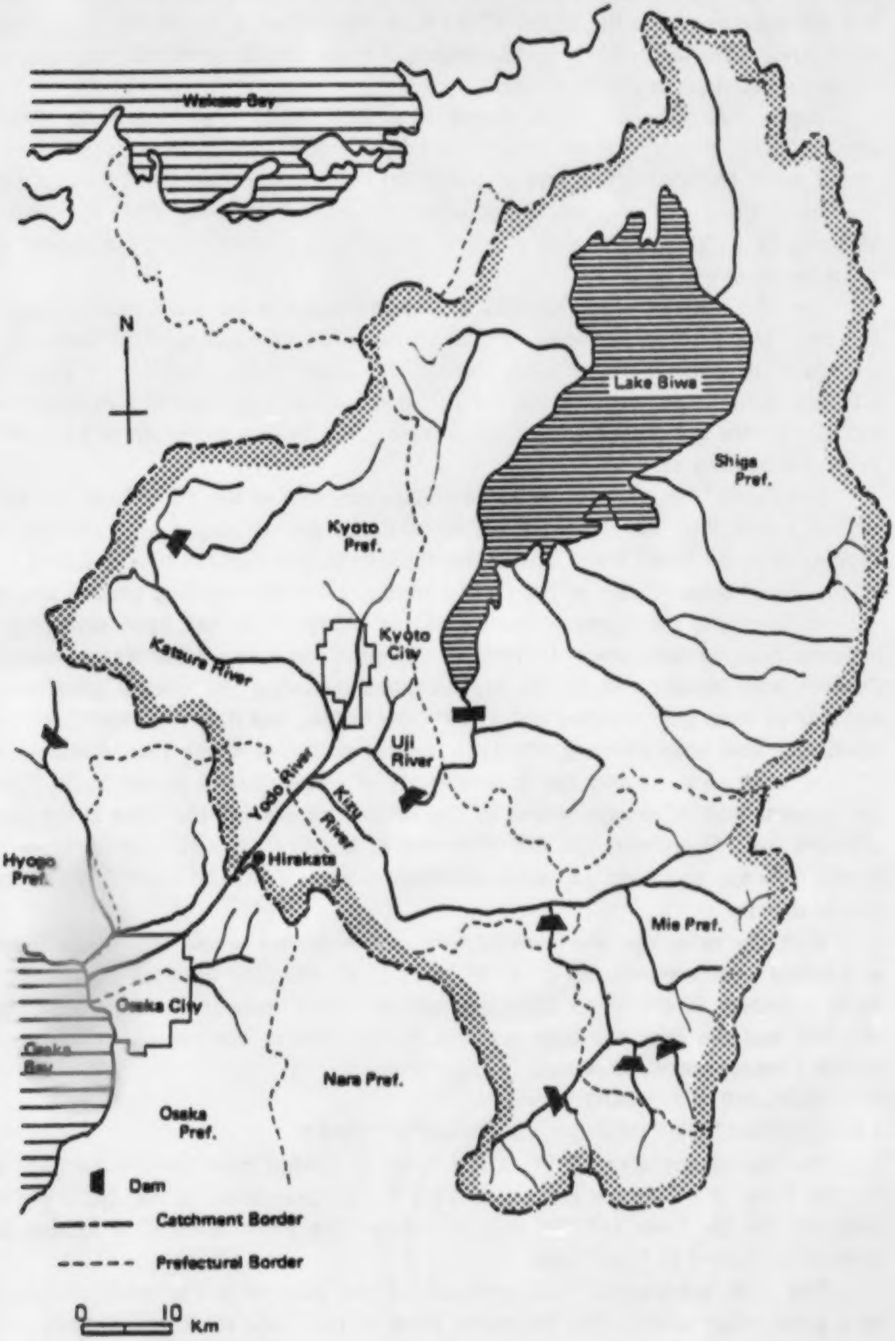
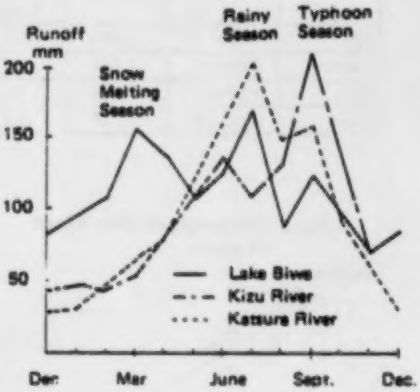

Wakasa Bay
N
Lake Biwa
Shiga Pref.
Kyoto Pref.
Kyoto City
Katsura River
Uji River
Yodo River
Kizu River
Hirakata
Hyogo Pref.
Mie Pref.
Osaka City
Osaka Bay
Nara Pref.
Osaka Pref.
Dam
Catchment Border
Prefectural Border
0
10
Km

Table 1 Catchments of Yodo River System

Name	Catchment Area Km^2	Length Km
Lake Biwa	3,848	–
Uji River	506	53.9
Kizu River	1,596	189.2
Katsura River	1,100	114.3
Main Stream of Yodo River	231	35.8
Total	7,281	393.2

Fig. 2 Monthly Flow of Yodo River System



sufficient water resources to the Yodo River: Namely, runoff of rain water in the three basins of the Yodo River is recording much quantity in the Lake Biwa basin for snowmelting period of March and April, the Katsura River basin for the rainy season of June and July and the Kizu River basin during the typhoon season of August and September, respectively (See Fig. 2).

Accordingly, these basins with their respective climatic features contribute to the stabilization of flow of the Yodo River by mutually covering their different points. Particularly, outflow from the Lake Biwa basin which occupies about half of the Yodo River basin has much advantage in river improvement and water utilization through the natural adjustment of Lake Biwa with its 680 km^2 in surface area and 27,500 million m^3 in Lake water capacity.

The Yodo River catchment basin also comprises of several basins geographically close to each other. At the time of flooding, a narrow portion of the respective basin makes a natural adjustment to attenuate peak flow and waters used for water supply, agriculture and industry in the upper and middle stream basin, are also recycled for use in the lower river basin.

Besides the above-mentioned natural advantages, flow control is being carried out at the dams of Takayama, Shorenji, Muro-o and Amagase and Seta River weir. As a result, the flow at Hirakata of the Yodo River is showing stabilization. High water and ordinary water volume are not much as compared with the important rivers in this country (See Fig. 3), but the volume of low water and drought water levels are big next to the Shinano River which runs in Nagano Prefecture.

The flow of various branch rivers of the Yodo River are shown in Table 2, but as compared in this table, catchment area in Lake Biwa occupies 53% of the Yodo River catchment basin, but contribution ratio to Hirakata flow is about 70%, showing the important role of Lake Biwa.

2.2 WATER UTILIZATION

The monthly flow of the Yodo River at Hirakata is comparatively stable through four seasons with 147.4 m^3/sec. – 467.2 m^3/sec. as shown in Fig. 4.

Table 2 Discharge of Branch Rivers of Yodo River System

Name of River	Name of Station	Discharge m³/sec.						
		Maximum	High Water	Ordinary Water	Low Water	Draughty Water	Minimum	Annual Average
Uji	Yodo	1,980	203	138	103	79	51	186
Katsura	Katsura	1,744	28	17	11	5	0	27
Kizu	Yawata	2,519	46	29	21	13	5	51
Yodo	Hirakata	5,228	282	197	148	105	66	264

1969–1978

Fig. 3 Discharges of Main Rivers in Japan

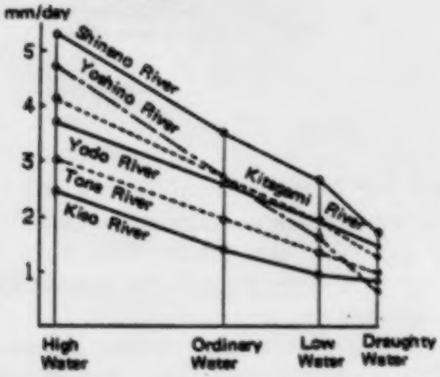
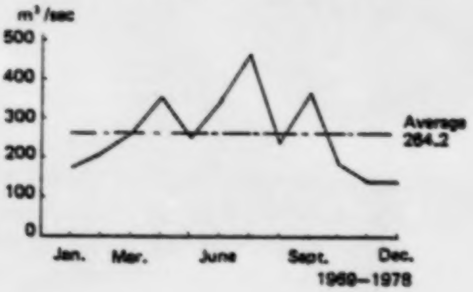


Fig. 4 Monthly Discharge of Yodo River (Hirakata)



As a result, the ratio of water utilization is high, and of about 10 billion m³ in annual volume of flow of the Yodo River, about 11% is used as water supply for general households, 4% as industrial water and 5% as agricultural water, or about 20% in water utilization ratio.

If maintenance water is included, the utilization ratio becomes about 45%. Thus, water in the Yodo River is being used very effectively as compared with 10% of the average utilization ratio elsewhere in this country. Since water demand is expected to further increase in parallel with economic development in the Kinki regional sphere, the ratio of dependence upon the Yodo River will become even higher.

As the total population and the water supplied population in the Yodo River basin are shown in Fig. 5, concentration of population was seen in the lower basin for 10 years in the upper period and in the middle basin and its vicinities for the same years in the latter period.

Water taken from the Yodo River as water resources is being supplied to almost all parts of Osaka Prefecture and some parts of Hyogo Prefecture while the water utilization population in these lowest basins is about 11 million. The water intake volume is shown in Fig. 6. As demand for water in the future is estimated as shown in Fig. 7, highly effective use of water resources is required along with the development of water resources.

Fig. 5 Population Change in Yodo River Catchment

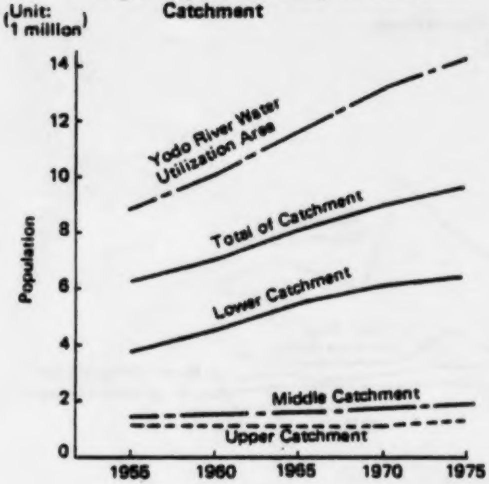


Fig. 7 Prediction of Water Demand for Hanshin Area

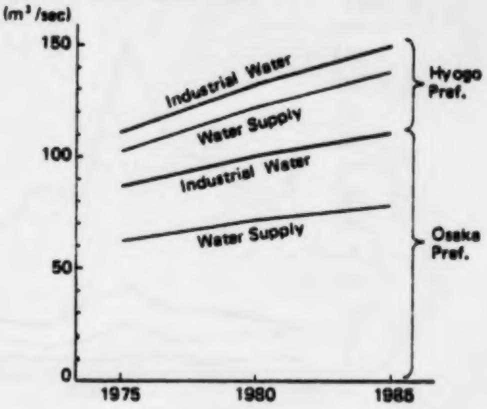
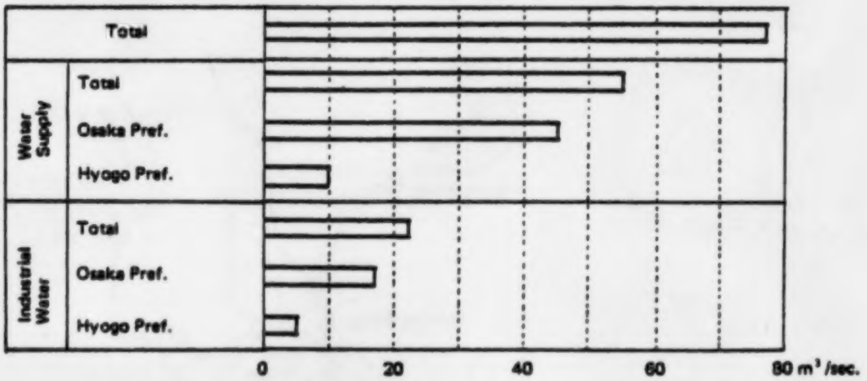


Fig. 6 Intake in Downstream of Yodo River (Maximum Intake) 1971



2.3 OUTLINE OF WATER QUALITY

The pollution of the water quality in the Yodo River has increased at a quick tempo since 1955 but the trend for improvement or stoppage of pollution is seen after 1969, which recorded the worst water pollution level. The outline of water quality according to rivers is described as follows: (See Fig. 8 and 9).

As to the Katsura River, it is hard to see change of water quality since around 1971 at Togetsu Bridge in the upper stream basin of the river because the river water at the bridge shows little artificial pollution with BOD (biochemical oxygen demand) making around 1 mg/ℓ. Miyamae Bridge which receives almost all wastewaters from Kyoto City marked BOD 23.4 mg/ℓ in 1969, the highest figure on record, but the BOD is dropped to 10 mg/ℓ since 1975.

As to the Uji River, water quality in its upper basin is close to that of Lake

Fig. 8 BOD Trend of Yodo River System

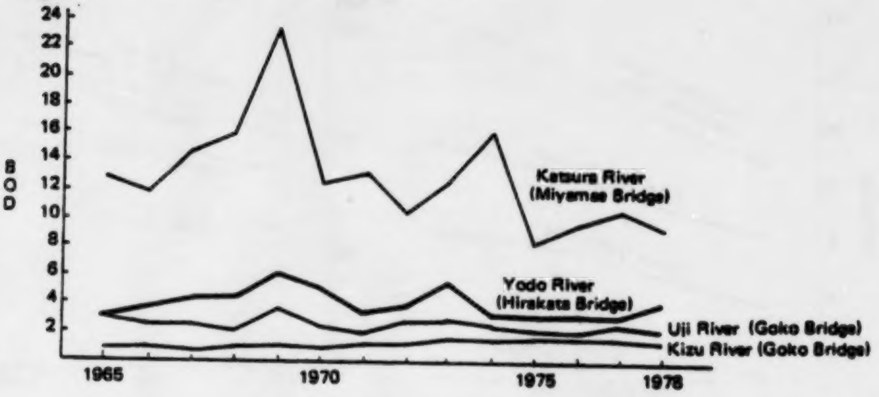
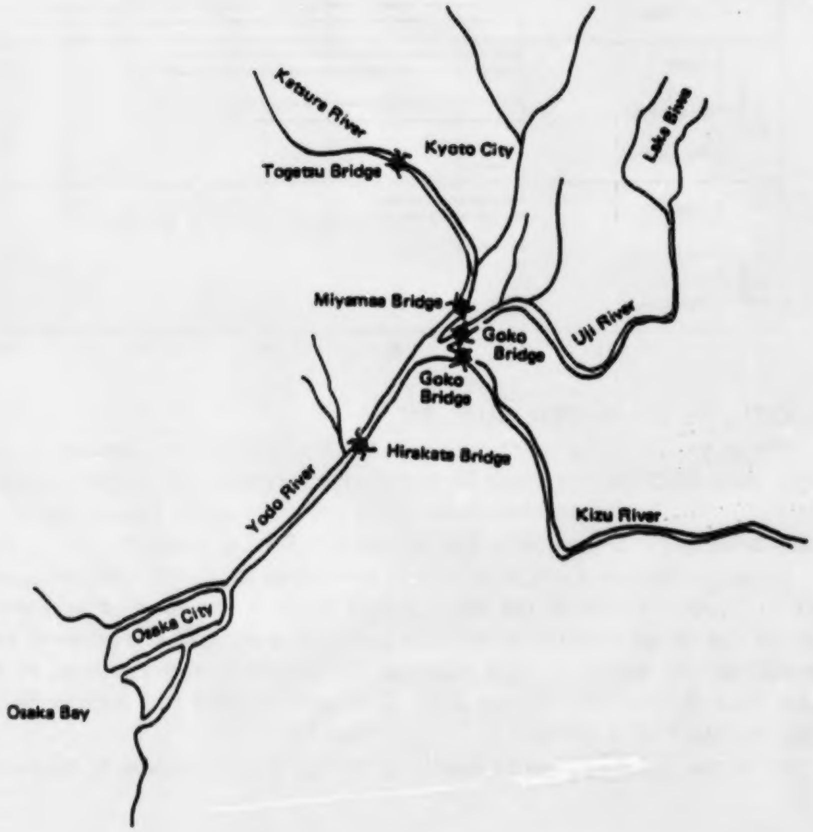


Fig. 9 Location of Sampling Point


Katsura River
Kyoto City
Togetsu Bridge
Lake Biwa
Miyamae Bridge
Goko Bridge
Uji River
Goko Bridge
Hirakata Bridge
Yodo River
Kizu River
Osaka City
Osaka Bay

Biwa, showing little change of BOD 1.5 mg/ℓ after 1975. Goko Bridge which receives some of the discharged waters of Kyoto City is also recording little change of BOD 2.4 mg/ℓ since 1975 after a high value of BOD 3.9 mg/ℓ in 1969.

As to the Kizu River, the water was clean with BOD marking less than 1 mg/ℓ before 1970 but the BOD gradually become higher in concert with the development in its basin, but is marking little change of BOD 1.6 mg/ℓ after 1973.

Water quality in the main stream of the Yodo River at Hirakata, involving the three biggest confluent rivers, is affected not only by wastewaters from Hirakata City areas on the left bank, but also by water qualities of the three branch rivers. The right bank of the river, affected by the Katsura River, is polluted remarkably by BOD 6.0 ~ 6.8 mg/ℓ from 1959 to 1973, showing a higher value than the BOD 4.5 ~ 5.5 mg/ℓ of the left bank but the difference of BOD concentration is becoming less in accordance with improvement of water quality in the Katsura River. As a result, the average BOD at Hirakata of the Yodo River was gradually improved from 6.2 mg/ℓ in 1969 as the highest figure to 3.2 mg/ℓ in 1975. The present condition of the water quality in these rivers is shown in Table 3.

Table 3 Water Qualities of Yodo River System

1978

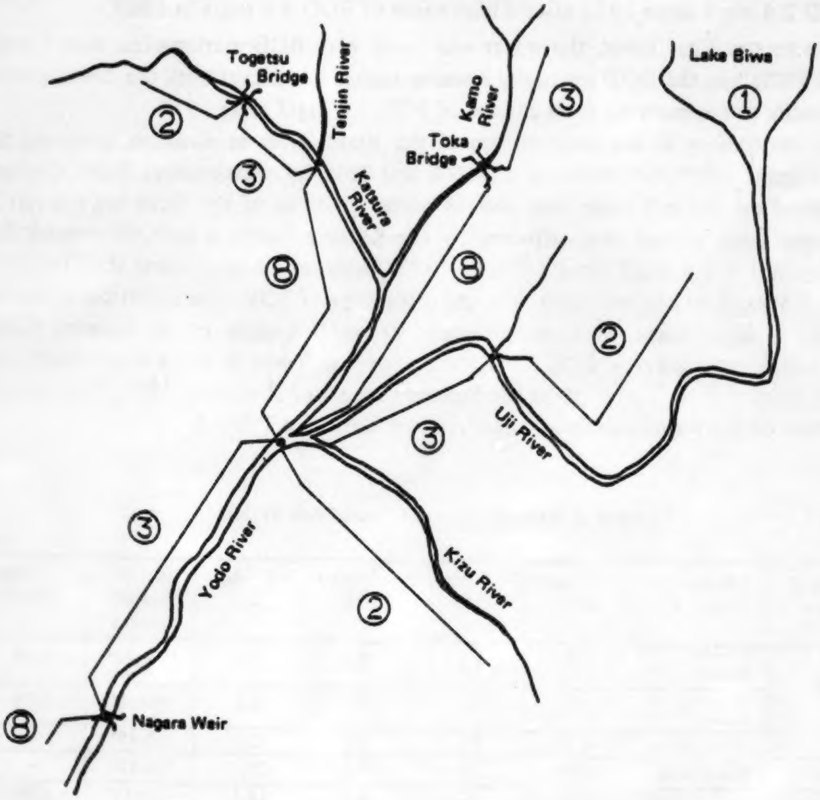
Name of River	Name of Station		mg/ℓ DO	mg/ℓ BOD	mg/ℓ COD	mg/ℓ SS	No. of Coliform Groups	mg/ℓ NH_4-N
Katsura	Togetsu Bridge		10.6	1.1	2.2	2.1	17.0×10^4	0.10
	Miyamae Bridge		7.0	9.4	13.7	26.5	3.0×10^5	3.73
Uji	Goko Bridge		9.2	2.3	3.3	10.2	2.9×10^4	–
Kizu	Goko Bridge		10.4	1.4	2.2	29.1	7.3×10^3	–
Yodo	Hira-Kata	Left	8.8	4.2	6.1	18.1	12.4×10^4	0.89
		Center	8.3	4.3	6.3	16.7	7.4×10^4	0.87
		Right	8.2	4.5	6.7	15.6	15.5×10^4	1.08

2.4 CONTROL BY LAWS

After the enforcement of the Basic Law for Environmental Pollution Control in 1967, the environmental standard for the Yodo River was established in September, 1970 (See Fig. 10). The Water Pollution Control Law was also enforced in 1970. As a result, control for wastewaters from the specified plants throughout the country and enforcement of punishment against violators of the laws were codified. The control for wastewaters from the specified facilities is handled by the Water Pollution Control Law and other prefectural ordinances but effluent from the sewage treatment plants are further regulated by the Sewerage Law. The effluent standards of sewage treatment plants in Kyoto City are shown in Table 4.

As the Seto Inland Sea Environmental Preservation Special Law and the Water Pollution Control Law were revised in 1978, the control on total load of COD (Chemical Oxygen Demand) discharged to the Seto Inland Sea, Ise Bay and Tokyo Bay, which are extensive closed water area, was adopted. Since Kyoto Prefecture

Fig. 10 Environmental Standard of BOD for Yodo River System (mg/ℓ)


Togetsu Bridge
Tenjin River
Kamo River
Lake Biwa
②
③
①
Toka Bridge
③
Katsura River
⑧
⑧
②
③
Uji River
③
Yodo River
Kizu River
②
⑧
Nagara Weir

had to accept application of the new control as a prefecture in the upper portion of the Seto Inland Sea, the total load of COD to be discharged from treatment plants in Kyoto City is also controlled. In the Seto Inland Sea, guidance will be given to the reduction of phosphorus and its compounds as a preventive measure against eutrophication.

The Kyoto Prefectural Government which has worked out a pollution preventive plan for the Yodo River Basin in 1972 according to the Basic Law for Environmental Pollution Control is now taking various countermeasures to achieve an environmental standard of water quality with the extension and construction of sewage works as a main pillar of these countermeasures. The extension and construction of sewage works are also required as a step of the total load control system for reducing pollutant substances from livelihoods. Thus, great attention is being paid to the sewerage systems in Kyoto City because the urgent extension and construction of the sewage works is eagerly desired.

Table 4 Effluent Standards of Treatment Plant

Items	Standard Value	Items		Standard Value
pH	5.8 ~ 8.6	PCB (mg/ℓ)		0.003 or Less
BOD (mg/ℓ)	Daily Ave. 20 or Less	N-Hexane Extracts	Mineral Oil (mg/ℓ)	5 or Less
SS (mg/ℓ)	Daily Ave. 70 or Less		Animal and Vegetable Fats (mg/ℓ)	30 or Less
Cliform Groups (per mℓ)	Daily Ave. 3,000 or Less	Phenol (mg/ℓ)		1 or Less
Cadmium (mg/ℓ)	0.05 or Less	Copper (mg/ℓ)		3 or Less
Cyanide (mg/ℓ)	0.5 or Less	Zinc (mg/ℓ)		5 or Less
Organic Phosphorous (mg/ℓ)	0.5 or Less	Iron (Soluble) (mg/ℓ)		10 or Less
Lead (mg/ℓ)	0.5 or Less	Manganese (Soluble) (mg/ℓ)		10 or Less
Hexivalent Chrome (mg/ℓ)	0.25 or Less	Chrome (mg/ℓ)		2 or Less
Arsenic (mg/ℓ)	0.25 or Less	Fluorine (mg/ℓ)		15 or Less
Total Mercury (mg/ℓ)	0.005 or Less	Nickel (mg/ℓ)		2 or Less
Alkyl Mercury (mg/ℓ)	ND	Boron (mg/ℓ)		1 or Less

3. SEWAGE WORKS PROJECT IN KYOTO CITY

3.1 PROJECT OUTLINE

The sewage works project in Kyoto City was begun in 1930 as an emergency unemployment countermeasure project. The city has been carrying out the project as one of the city planning projects since 1934. As the first treatment plant in Kyoto City, Kisshoin treatment plant (with treatment capacity of 7,800 m^3/day by activated sludge process) begun operation in 1934, followed by the operation of the Toba treatment plant (with treatment capacity of 78,000 m^3/day by activated sludge process) in 1939. The construction work of sewerage systems was suspended due to the Second World War in 1943, but the extension and construction of the sewage works had so far been made in an area of 1,342 ha in addition to the above-mentioned two treatment plants.

The project was resumed in 1949 and in 1956 the Long Range Plan for Construction of Sewage Works in Kyoto City was worked out to fully accelerate the project. On the other hand, the central government worked out its five-year plan starting in 1963 with the purpose of carrying out the extension and construction of sewage works. Its investment was rapidly expanded in one plan after another.

The Kyoto City Office is also accelerating a project based on its plan to meet the five-year plan of the central government. Kyoto City is positively taking the steps for the extension and construction of the sewage works as one of the important policies for an international sightseeing city and also for the security of water resources in the lower stream basin of the Yodo River to prevent the pollution of river water quality. The extension and construction of sewage works are shown in Table 5.

The sewage works project is now under way according to the fourth five-year plan for the fiscal years 1976–80 for the construction of sewage works by the central government. The five-year plan of Kyoto City called for the expenditure of ¥142 billion for the sewage works projects with the design to see that the treatment area is extended by 2,283 ha to 7,080 ha; treatment capacity is increased by 242,000 m³/day to 1,005,000 m³/day and the ratio of served population against total population is set from 47% to 61% (Table 6). The construction project is shown in Table 7.

Total designed sewer area of Kyoto City which is equal to the proposed future urbanized area (16,221 ha) is divided into six treatment districts as shown in Table 8 and Fig. 11. By 1990, all of the present urbanized area is scheduled to be served by sewerage systems.

Table 5 Sewage Works Status – April 1979

Total City Area	61,061	ha
Urbanized Area	14,887	ha
Total City Population	1,458,019	Persons
Service Area	5,857	ha
Percentage to Urbanized Area	39.3	%
Service Population	758,110	Persons
Percentage to Total Population	52.0	%
Sewer Length	1,534,518	m
Treatment Capacity	957,500	m³/day
Toba Plant	750,000	m³/day
Kisshoin Plant	125,000	m³/day
Fushimi Plant	82,500	m³/day

Table 6 Summary of Five-Year Plan (FY 1976–1980)

Total Cost	¥142	Bil.
For Old Urban Area	¥130	Bil.
For New Urban Area etc.	¥ 12	Bil.
Project Target (At End of 1980)		
Service Area	7,080	ha
Percentage to Urbanized Area	48	%
Service Population	918,000	Persons
Percentage to Total Population	61	%
Sewer Length	2,012	Km
Treatment Capacity	1,005,000	m³/day

Table 7 Sewage Works of Kyoto City

Item \ Year		1971	1972	1973	1974	1975	1976	1977	1978	1979
Cost										
Total Cost	(¥1 Mil.)	7,505	8,275	13,900	16,047	24,918	23,659	26,187	33,683	40,000
Sewer	(¥1 Mil.)	3,240	4,745	7,715	11,874	14,812	14,962	20,960	25,326	32,920
Plant	(¥1 Mil.)	4,265	3,530	6,185	4,173	10,106	8,697	5,227	8,357	7,080
Service Area (ha)	Increase in Year	187	217	392	221	200	254	356	450	788
	Total at End of Year	3,767	3,984	4,376	4,597	4,797	5,051	5,407	5,857	6,645
Sewer Length	(Km)	997.5	1,066.8	1,131.3	1,199.1	1,267.4	1,331.4	1,428.9	1,534.5	1,758.2
Service Population		616,060	651,650	681,690	688,140	693,700	700,260	726,830	758,110	808,400
Treatment Capacity (1,000m³/day)	Increase in Year	72.5	117.5	–	–	45	45	59.6	90	47.5
	Total at End of Year	600.4	717.9	717.9	717.9	762.9	807.9	867.5	957.5	1,005
		← 3rd 5-Year Program →					← 4th 5-Year Program			

Table 8 Overall Program

Name of Sewer District	Sewer		Plant		Remarks
	Service Area	Type of Collection System	Treatment Capacity	Type of Treatment Process	
Toba	8,160 ha	Combined, Separate	m³/day 1,225,000	Activated Sludge Process	
Kisshoin	587	Combined	165,000	"	Oxygen Aeration Process for 80,000 m³/day
Fushimi	2,171	Combined, Separate	238,000	"	
Yamashina	1,984	Separate	163,000	"	
Katsura	2,800	Separate	–	–	Sewerage System for Right Bank Catchment of Katsura River
Mukaijima	519	Separate	–	–	Sewerage System for Left Bank Catchment of Uji River
Total	16,221		1,791,000		

Note: Sewerage System for Right-Bank Catchment of Katsura River
Planned Area – A part of Kyoto City, Muko City, Nagaokakyo City, Oyamazaki Town; 5,088 ha
Rakusai Purification Center; 427,000 m^3/day. Partially operated on Oct. 18, 1979
Sewerage System for Left-Bank Catchment of Uji River – Plan under way.

3.2 IMPROVEMENT OF WATER QUALITY IN YODO RIVER

When pollution load flowing into the main stream of the Yodo River in 1970 was compared to other rivers, the Katsura River marked 60%, the Uji River 36% and the Kizu River about 4%. The ratio for each prefecture recorded at Hirakata point in the main stream of the Yodo River is shown in Fig. 12. Kyoto City occupies its great weight in water quality pollution in the Yodo River with about 55% in all basins and about 69% in midstream basin.

In Kyoto City, an increase of wastewaters from industrial factories and general households with the rapid industrial development (See Fig. 13) and increase of population and promotion of livelihood standards (See Fig. 14) have brought an increase of pollutant substances, causing pollution problems in the rivers due to the delay in extension and construction of sewage works. On the other hand, treatment plants faced a lack of treatment capacity with the increasing inflow of sewage and treatment efficiency was lowered due to the increased wastewaters from factories, markedly worsening effluent quality for a time.

There are many dyeing plants in Kyoto City (with about 24% in industrial shipment and about 55% in the total number of factories), greatly affecting influent quality at the treatment plants. The overload of water volume and water quality due to the discharge of nightsoil into the sewer caused insufficient treatment. Since instant effect on improvement of water quality of the river can be obtained by expanding treatment capacities at first, Kyoto City has decided to exert its efforts particularly for the construction of treatment facilities. About 49.3% of the total project cost was invested for construction of treatment plants to increase the treatment capacities from fiscal 1966 to 1972.

Fig. 11 Sewer District of Kyoto City

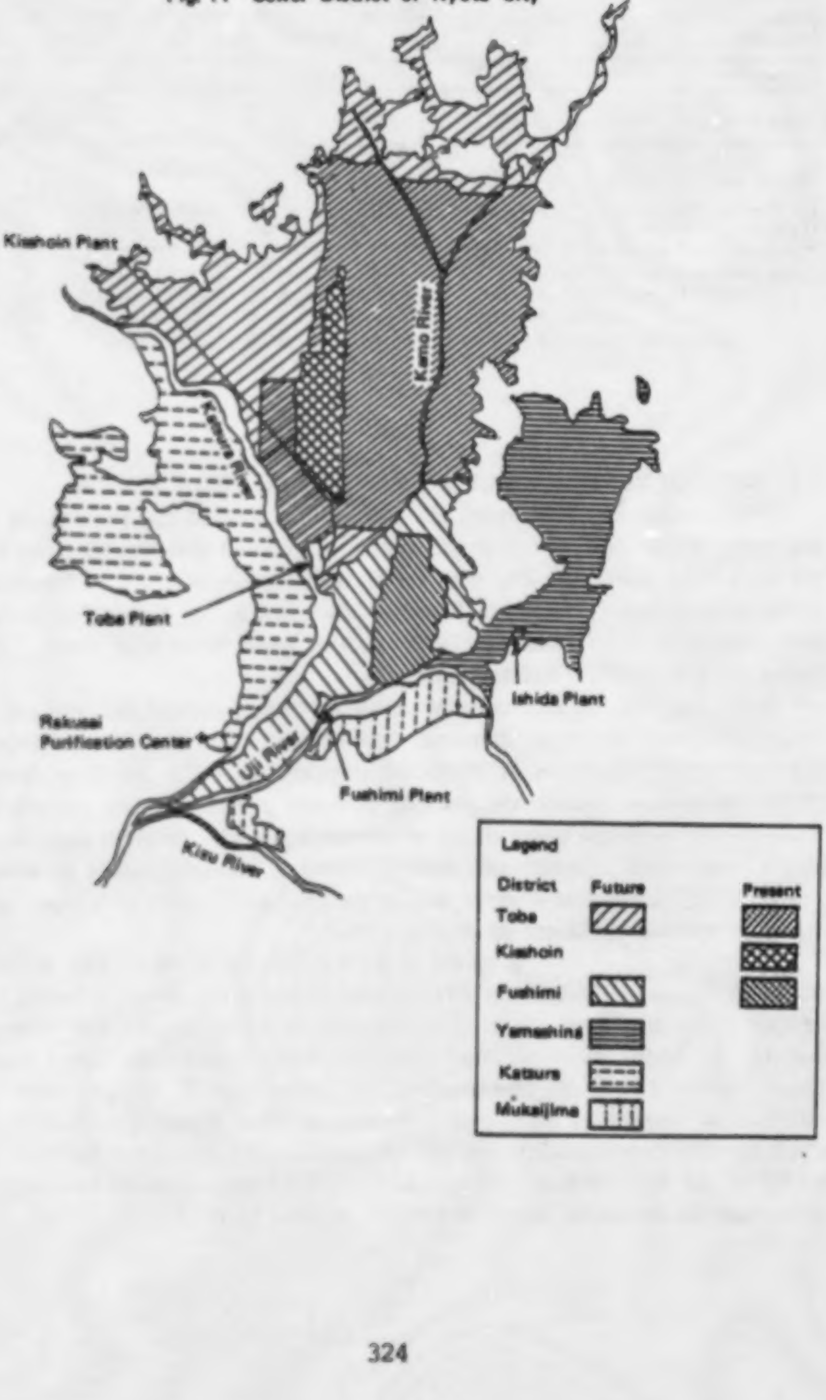
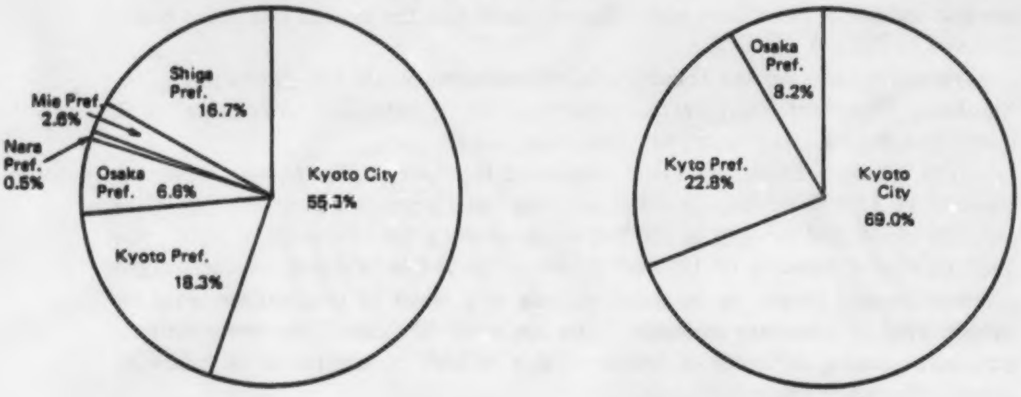

Kishoin Plant
Kamo River
Katsura River
Toba Plant
Ishida Plant
Rakusai Purification Center
Uji River
Fushimi Plant
Kizu River
Legend
District
Future
Present
Toba
Kishoin
Fushimi
Yamashina
Katsura
Mukaijima

Fig. 12 Breakdown of Pollutant Load Discharged into Yodo River by Prefecture At Hirakata Point in 1970



Whole Catchment

Middle Catchment

Fig. 13 Yearly Change of Industrial Products Shipped

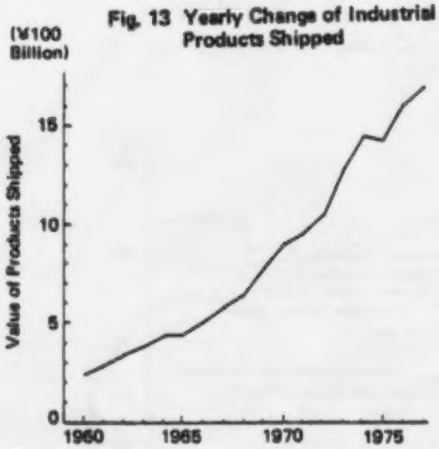
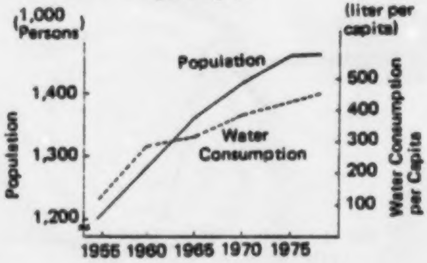


Fig. 14 Population & Water Consumption per Capita



Kyoto City started the discharge of nightsoil into the sewer from around 1950. In order to solve the adverse effect caused by the discharge of nightsoil, the city office built digestion tanks (capacity–415 kℓ/day) in the Toba Treatment Plant compound in 1966 and installed storage facilities in 1968 at two nightsoil discharge depots in the city to discharge nightsoil uniformly into the sewer during the night period of low inflow load to the treatment plant.

On the other hand, Toba Treatment Plant extended its facilities to meet the increasing quantity of inflow sewage and maintain the water quality, resulting in the gradual improvement of effluent quality and upgrading the water quality to the level that is a little better than effluent standards since 1972.

The treatment capacity, inflow sewage quantity, discharged nightsoil quantity and effluent quality at the Toba Treatment Plant are shown in Fig. 15 and the present condition of influent and effluent qualities at the various treatment plants is shown in Table 9.

As the process for the construction of treatment plants was shown in Fig. 16, the Toba Treatment Plant has been carrying out its extension work since 1959, currently reaching 750,000 m³/day treatment capacity.

The Kisshoin Treatment Plant completed the A-line facilities with treatment capacity of 85,000 m³/day in 1967 and also built B-line facilities with treatment capacity of 40,000 m³/day in 1977 after dismantling the old facilities, raising the total treatment capacity of 125,000 m³/day. The B-line facilities use the oxygen aeration process judged to be most suitable as a result of comparative tests for various kind of treatment methods. There are many factories in the sewer district, especially causing difficulty of treatment due to high concentration of inflowing sewage affected by dyeing effluents.

The Fushimi Treatment Plant has completed facilities with a treatment capacity of 27,500 m³/day and started the operation in 1972. The present treatment capacity is 82,500 m³/day.

Fig. 15 Relationship between Effluent BOD and Treatment Capacity at Toba Plant

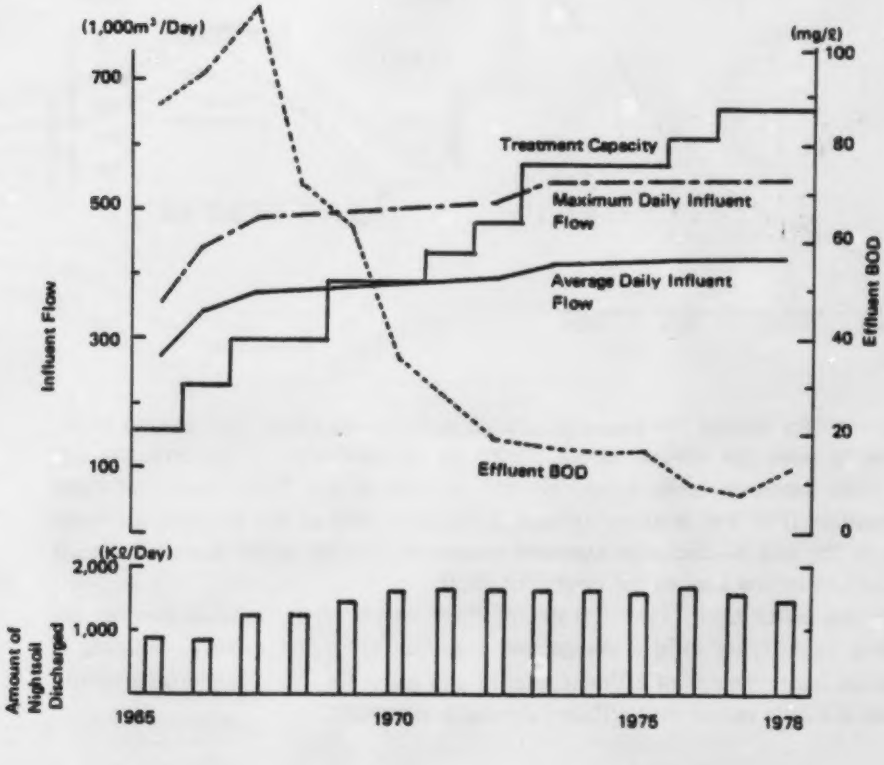
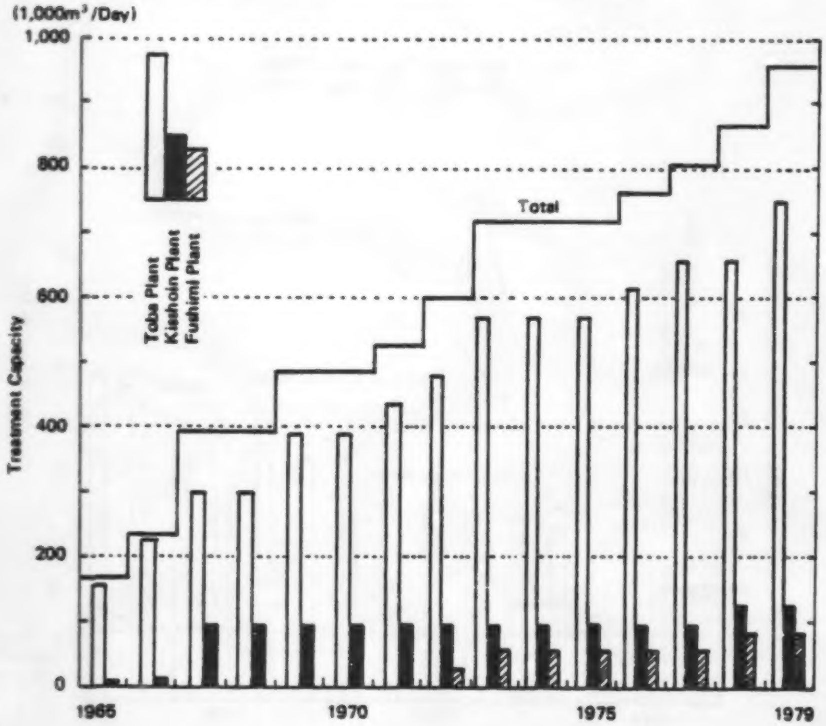


Table 9 Characteristics of Influent and Effluent At Each Plant (mg/ℓ)

	BOD	SS	NH₄-N	T-P	Anion Surfactant
Toba Plant					
Influent	223.6	152	13.30	3.52	5.4
Effluent–Nishi Takase River	11.4	13	9.67	2.09	0.3
Effluent–Katsura River	11.4	7	9.67	1.26	0.3
Kisshoin Plant					
Influent–A-Line	137.5	99	8.92	2.94	7.1
Influent–B-Line	212.5	161	18.27	3.14	5.3
Effluent	13.8	8	9.59	1.17	0.7
Fushimi Plant					
Influent	151.9	99	5.49	2.98	3.5
Effluent	12.1	8	1.20	1.07	0.3

Fig. 16 Breakdown of Treatment Capacity by Plant



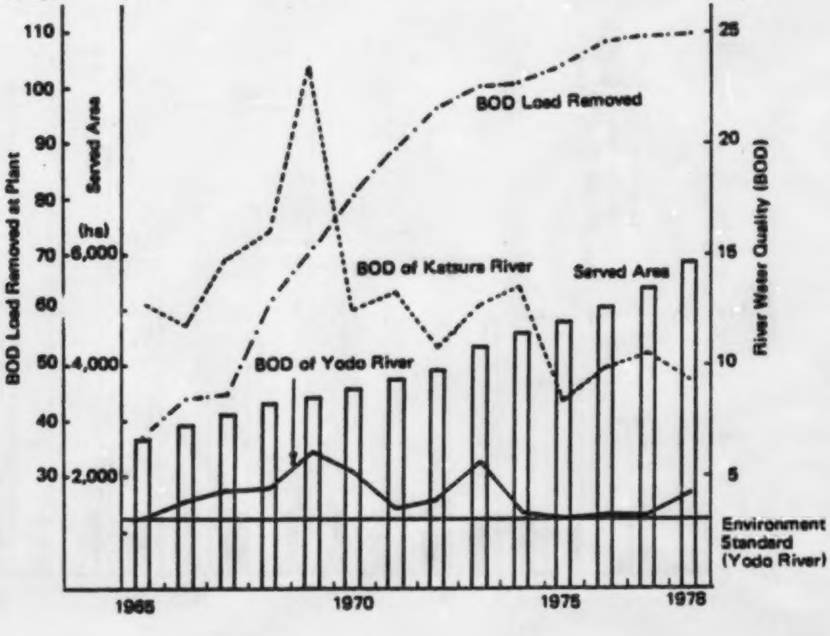
In addition to three plants, the new Ishida Treatment Plant is now under way. The plant is expected to start operation with a treatment capacity of 20,000 m³/day by the end of 1980. If the plant starts its operation, all the treatment plants in Kyoto will be operated.

As mentioned, the extension and improvement of the Toba Treatment Plant, which treats most of the sewage in Kyoto City, has contributed to the improvement of the sewage works through the improvement of treatment and extension of sewer district. As a result, pollutant substances, discharged from the city, have remarkably decreased. The water quality in the Katsura River was 23.4 mg/ℓ in the highest BOD, but it became 8 to 10 mg/ℓ recently. The water quality of the Yodo River was 6.8 mg/ℓ at its worst but recently became 3 mg/ℓ, almost reaching the environmental standard (See Fig. 17).

The water quality of the Yodo River aggravated in 1973, 1977 and 1978 but this was due to the fact that these years were water shortage years, aggravating flow conditions in various rivers. When the average annual discharge in these years is compared with those of the last 10 years, 1973 was about 77% and 1977 was about 74% while 1978 decreased to about 55%.

The BOD load surveyed at Hirakata in the main stream of the Yodo River was 54.7 t/day on annual average in 1977 and 52.3 t/day in 1978, remaining little change. The effect of water quality improvement in the Yodo River by the exten-

Fig. 17 Relationship between River Water Quality and Sewage Works



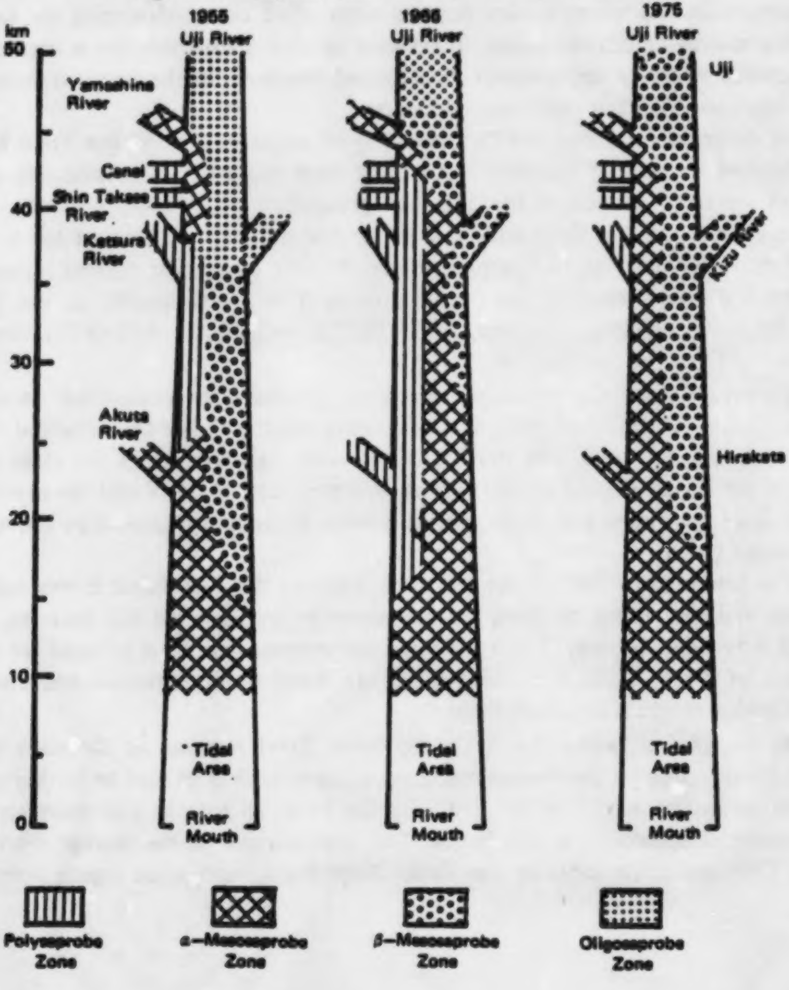
sion and construction of the sewage works in Kyoto City reflects the survey for biological water quality pollution. Fig. 18 shows pollution of water quality according to the index for judgment of biological water quality at 10 years intervals since 1955.

According to this, water pollution in the Yodo River turned to worse during the decade from 1955 to 1965, but the water quality in 1975 is believed to have dropped to that in 1955 due to remarkable reduction of pollutant substances discharged from the Katsura River, Shin Takase River and canal as a result of the extension and construction of sewage works in Kyoto City.

4. CONCLUSION

The Yodo River flows through six prefectures as the most important water resource in the Kinki regional sphere and its water utilization population is about

Fig. 18 Biological Water Quality Pollution Maps



14 million. But since Kyoto City is situated in the midstream in the Yodo River basin, it faced water quality problems from an early period as important issues for water utilization for the downstream basin and the reduction of pollutant substances from Kyoto City became an urgent issue.

The water quality pollution in the Yodo River was improved to the extent that the environmental standard relating to human health on noxious substances such as cadmium and mercury was maintained due to strict control over water quality in wastewaters discharged from factories, plus the construction of the sewage works in Kyoto City and the environmental standards relating to living environment such as BOD etc. were also almost achieved.

A new problem, however, is the trend to increase ammonia nitrogen in the river. Ammonia nitrogen often reaches a considerably high concentration at the time of low flow or at low water temperature in the river.

The destination of discharge in the Yodo River is the Seto Inland Sea which is extensive closed water bodies but red tides often occur, damaging the fishing industry and living circumstances. In respond to such a situation, the sewage works are required to study employment of advanced treatment as the occasion demands, in addition to upgrading treatment efficiency.

In order to carry out the improvement of water quality in the Yodo River, the reduced amount of pollutant substances from the cities is needed. Since the ratio of served population to the total city population is 52% at the present, Kyoto City is considering the extension of the area for the construction of the sewage works by a secondary treatment process for the time being. The present secondary treatment showed effective for the improvement of water quality in the Yodo River by removing organic matters such as BOD as well as 20 ~ 40% of the nitrogen and 30 ~ 50% of the phosphorus.

However, since the advanced treatment processes are considered necessary in the future, research and study are now being conducted for the advanced treatment technologies. In Kyoto Prefecture, the sewerage systems in the right bank basin of the Katsura River started operation from October, 1979 and the extension of the sewerage systems is under way in Kyoto, Muko and Nagaokakyo cities and Oyamazaki Town.

The sewerage project of the left bank basin of the Kizu River is also making progress and the survey to complete the sewerage plan .for the left bank basin of the Uji River is under way. Since the sewerage systems have been planned for most all areas of Kyoto Prefecture along the Yodo River Basin, extensive construction of the sewage works is expected soon.

At the present, water quality in the Yodo River remains on the same level, so the construction of the sewage works on a larger scale is needed to further carry out the improvement of water quality in the river, to achieve and maintain the environmental standards in the future. The construction of the sewage works in Kyoto City and other cities in the Yodo River Basin remains an urgent problem.

Seventh US/JAPAN Conference
on
Sewage Treatment Technology

PRACTICAL APPLICATIONS FOR REUSE OF WASTEWATER

May 21, 1980
Tokyo, Japan

Takeshi Kubo

Vice President,
Japan Sewage Works Agency

PRACTICAL APPLICATIONS FOR REUSE OF WASTEWATER

1. An Outline of Water Resources in Japan 333

2. Water Utilization and Future Demand 339

3. Future Demand and Supply Program for Water Resources 344

4. Strengthening the Water Quality Management Program in the Basin 348

5. Conservation of Water 351

6. Reuse of Municipal Wastewater Effluents 358

    6.1 Present Status of Municipal Wastewater Effluents Reuse 358

    6.2 Problems to Be Examined and Studied in Direct Reuse of Municipal Wastewater Effluents 360

        1. Water Quality Standard for Direct Reuse 360

        2. The Economics of Direct Reuse of Municipal Wastewater Effluents 360

        3. Energy Consumption in the Direct Reuse of Municipal Wastewater Effluents 361

        4. Water Rights 361

        5. Psychological Problems Resulting from the Direct Reuse of Municipal Wastewater Effluents 362

1. AN OUTLINE OF WATER RESOURCES IN JAPAN

Japan is a long and narrow island country in which 70% of the land is mountainous and each river basin drainage area is relatively small. For example, the longest river in Japan, the Tone River is only 322 km in length. Most rivers flow rapidly from the mountains to the sea. Rain water comes down and runs swiftly into the sea. This process takes only a few days, and consequently the water quality of rivers in Japan has been maintained quite well. Historically, the Japanese people have enjoyed the nation's natural beauty, describing it with phrases such as "purple mountains and clean waters."

Under these circumstances the numbers of times water is reused in the course of its flow downstream in the river basin is quite small in comparison with water reuse in continental rivers.

Statistics calculated from data gathered for 18 years from 1956 to 1973 at 1,260 rainfall gauging stations located throughout the country shows that average annual rainfall in a normal year is 1,788 mm. It is 1,480 mm and 2,131 mm for dry and wet year respectively. The total amount of annual rainfall over the nation's total area of 377,484 km^2 for normal, dry and wet years are 674.9 $bil.m^3$, 558.7 $bil.m^3$ and 804.4 $bil.m^3$, respectively. If we subtract the amount of evaporation loss from these figures we can calculate the absolute maximum water resources available as shown in Table-1.

The annual loss of water due to evaporation is known to remain at almost the same level every year, but it is also known that the evaporation loss in mountain areas and in the flat areas differ.

The National Land Agency divides the availability of water resources into the following three classes.

Class 1 Amount of rainfall in mountainous regions where water can be stored in impounding reservoirs and maintained in good quality for use downstream.

Class 2 Amount of rainfall in flat inland regions where water can be used for agricultural irrigation and other purposes. This water is less useful than that found in mountainous regions.

Class 3 Amount of rainfall in narrow coastal regions. This water may possess the least utility.

Out of nation's total land area, the proportion of mountainous, flat inland and coastal regions are respectively 70%, 18% and 12%. The maximum available water resources in Japan can be divided into three classes, as shown in Table-2. Judging from this data it can be said that a fairly large amount of water (around 70%) is highly available. We, however, must consider the regional distribution of water, because the amount of maximum available water differs from region to region.

When we look at the regional distribution of annual rainfall (available), it varies greatly by region, as shown in Table-3 and Fig. 1. Evaporation loss also varies by region according to whether it is mountainous or flat, as shown in Table-4.

The major seasons with substantial rainfall in Japan differ from region to region. Usually they are early summer (called the Baiu season) and early autumn (called the typhoon season). But in Hokkaido, Tohoku and Hokuriku on the Japan Sea coast, winter brings heavy snowfalls. Actually there is no Baiu season in Hokkaido region. In the Tokyo area, 20% of the annual rainfall comes during the typhoon season, with another 20% falling during Baiu season. Both midsummer and winter are relatively dry. This means that rainfall is concentrated in short wet seasons. There is no constant rainfall during the year. This makes it difficult to utilize water resources effectively.

The maximum water resources available in each region can be calculated by using data on the annual rainfall (available) and evaporation loss, as shown in Table 3 and 4.

From the view point of water usage, the maximum water resources available per capita can also be calculated. Particularly data in a dry year may be useful. This is shown in Table-5.

Table-1 Water Resources in Japan

	Annual Rainfall (mm)	Total Amount of Rainfall (bil.m^3)	Maximum Available Water Resources (bil.m^3)
Dry Year	1,480	558.7	333.3
Normal Year	1,788	674.9	449.4
Wet Year	2,131	804.4	579.1

Note: 1. Calculated from data gathered at 1,260 rainfall gauging stations throughout the country during the 18 years from 1956 to 1973

2. Dry year: The year when the second lowest rainfall was registered during the 18-year period.

3. Normal year: The 18 year average

4. Wet year: The year when the second highest rainfall was registered during the 18-year period.

Table-2 Availability Classification of Water Resources in Japan

	Class 1 (bil.m^3)	(%)	Class 2 (bil.m^3)	(%)	Class 3 (bil.m^3)	(%)	Maximum Available Water Resources (bil.m^3)	(%)
Dry Year	236.9	71.1	58.1	17.4	38.3	11.5	333.3	100.0
Normal Year	310.8	69.2	81.9	18.2	56.7	12.6	449.4	100.0
Wet Year	394.4	68.1	107.6	18.6	77.0	13.3	579.1	100.0

..

Table-3 Annual Rainfall (available) by Region in Japan

Region	Annual Rainfall (available) (mm)		
	Dry Year	Normal Year	Wet Year
Hokkaido	590	764	956
Tohoku (The Pacific Side)	1,153	1,439	1,724
Tohoku (The Japan Sea Side)	639	898	1,148
Kanto	748	1,023	1,305
Hokuriku	1,633	2,026	2,429
Tokai	1,379	1,780	2,214
Kinki	959	1,283	1,624
Chugoku	868	1,192	1,607
Shikoku	1,021	1,439	2,043
Kyushu	924	1,406	2,000
Okinawa	267	1,127	1,791
Whole Country	883	1,191	1,534

Table-4 Evaporation Loss by Region

Region	Loss in Mountainous Regions (mm/year)	Loss in Flat Regions (mm/year)
Hokkaido	400	560
Tohoku (The Pacific Side)	400	560
Tohoku (The Japan Sea Side)	600	840
Kanto	450	630
Hokuriku	650	910
Tokai	450	630
Kinki	600	840
Chugoku	600	840
Shikoku	650	910
Kyushu	700	980
Okinawa	-	980

Fig. 1. Map of Japan by Region

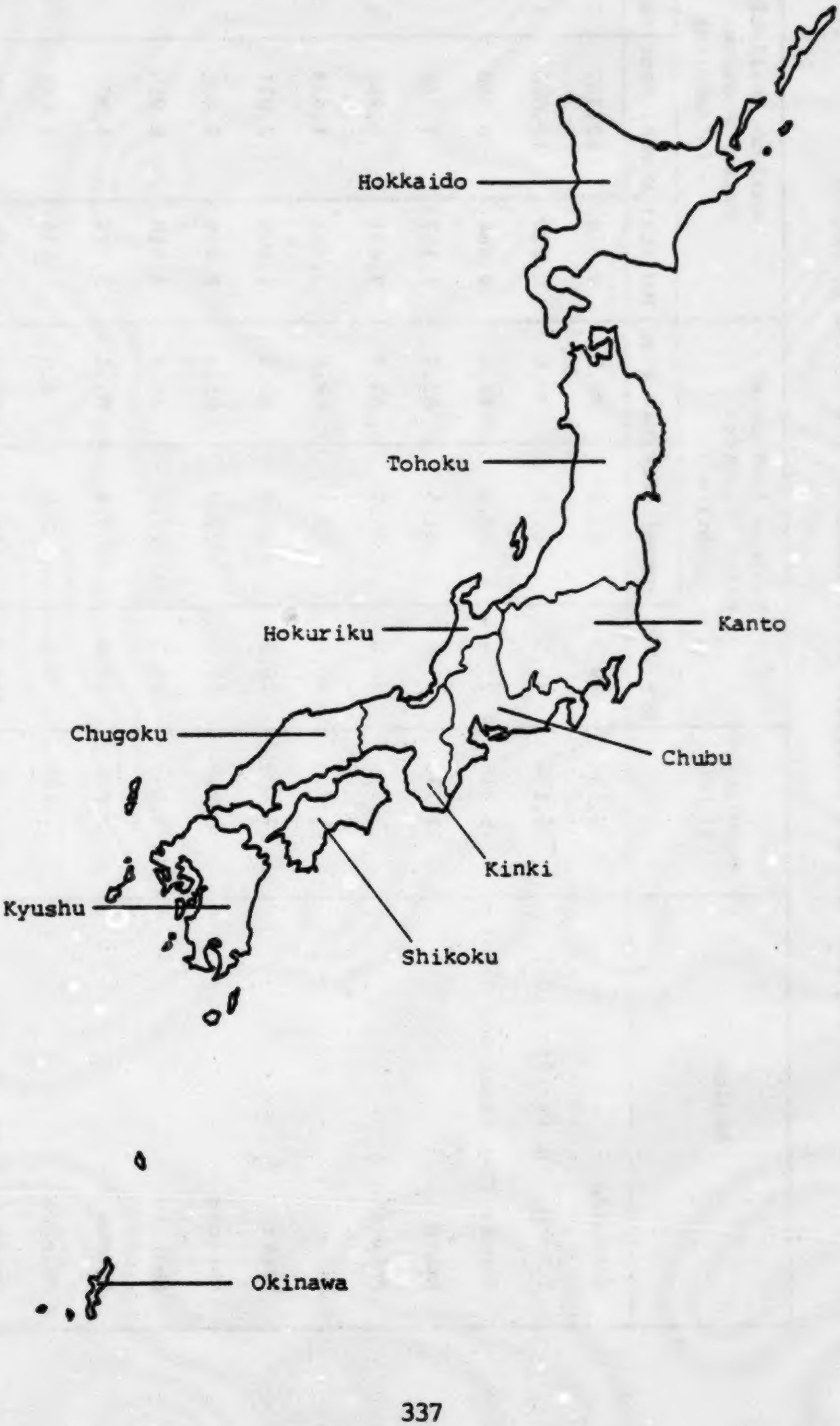

Hokkaido
Tohoku
Kanto
Hokuriku
Chugoku
Chubu
Kinki
Kyushu
Shikoku
Okinawa

Table-5 Maximum Available water Resources per capita by Region

Region	Population (1,000)	Maximum Available Water Resources (bil.m^3)			Maximum Available Water Resources (m^3/capita)		
		Dry Year	Normal Year	Wet Year	Dry Year	Normal Year	Wet Year
Hokkaido	5,184	49.3	63.8	79.9	9,510	12,307	15,413
Tohoku (The Pacific Side)	6,176	49.5	61.8	74.0	8,015	10,008	11,982
Tohoku (The Japan Sea Side)	5,216	23.4	32.9	42.0	4,486	6,308	8,052
Kanto	32,776	37.6	51.5	65.7	1,167	1,549	2,039
Hokuriku	2,776	20.6	25.7	30.7	7,421	9,258	11,059
Tokai	11,778	40.3	52.1	64.8	3,422	4,424	5,502
Kinki	17,402	26.2	35.0	44.3	1,506	2,011	2,546
Chugoku	6,997	27.7	37.9	51.2	3,959	5,431	7,317
Shikoku	3,904	19.2	27.0	38.4	4,918	6,918	9,836
Kyushu	12,071	38.9	59.2	84.1	3,223	4,904	6,967
Okinawa	945	0.6	2.5	4.0	635	2,645	4,233
Whole Country	104,665	333.3	449.4	579.1	3,185	4,294	5,533

The maximum water resources available in a dry year for the Kanto region (including the Tokyo Metropolitan area) is the lowest in Japan, registering 1,167 m^3/capita/year. The value of maximum available water resources is a theoretical one. In reality, practical available water resources have been mainly limited by the economic base.

Actually, water consumption in 1975 was estimated at about 90 billion cub. m. This corresponds to 27% of the maximum water resources of 333.3 billion cub. m. available in a dry year.

When we consider that riverflow is reduced to less than one-third the annual average maximum available water resources during dry seasons, it can be said that this figure appears to be rather a high value.

2. WATER UTILIZATION AND FUTURE DEMAND

In the utilization of water in Japan, the agricultural sector has historically received.top priority and special concessions. Because the staple food of the Japanese people has been rice and paddy fields have been developed gradually throughout the country since 200 B.C., particularly during the farming season sufficient irrigation water has been demanded by farmers. But most agricultural water has long been used in accordance with traditional practices and, therefore, agricultural water consumption has not been fully analyzed, as compared with other water uses.

The agricultural land in the suburbs of large cities has been considerably reduced in the past twenty years because of the rapid urbanization of these areas. Demand for agricultural water usage has been decreasing rapidly in these areas. But the demand for municipal and industrial water has increased quickly in the same period thanks to urbanization and industrialization.

Urbanization has been swift. Structural change in industries has occurred and at the same time population has commenced to move from agricultural villages to cities. It is anticipated that this trend will continue. As shown in Table-6, the population of urban areas was 43.7% of the total population of Japan in 1960. This figure increased to 48.1% in 1965 and 53.5% in 1970. National Land Agency estimates that this figure will increase to 71.5% by the year 2000.

According to this estimate by the National Land Agency, the Ministry of Construction announced a survey of "Present and Future Demand for Water". As shown in Table-7, this estimates water demand by region. In 1975, 87.6 billion cub. m. of water was used in Japan. Out of this figure, 12.3 billion cub. m. of water was used for municipal purposes (14% of the total water used); 18.3 billion cub. m. was for industrial use (20.9%); and 57.0 billion cub. m. was for agricultural use (65% of the total).

The estimate of future water demand in Japan by 1985 is 114.5 billion cub. m. Out of this figure, 18.7% is for municipal use, 25.6% is for industrial use and 55.7% is for agricultural use. Municipal water demand has been stimulated by the rise in the standard of living, urban population growth and larger city size. This has led to an increase in per capita consumption, as shown in Table-8. Municipal water demand is expected to continue growing in the future. Water demand for industrial uses has increased since 1960 as top priority was placed on heavy industry.

Most heavy industries have been located in coastal areas. They have tried to draw their water from groundwater sources. From around 1950 the utilization of groundwater for industrial purposes increased sharply. But exessive drawing of groundwater, particularly for industrial purposes, has caused the following problems;

(1) The groundwater table in coastal industrial areas was lowered and the groundwater itself was invaded by salt water flows.

(2) Land subsidence occurred on a large scale due to the exessive drawing of groundwater.

Table-6 Increase of Urban Population

	Total Population (thousands)	Urban Population (thousands)	Urban Population / Total Population (%)	Growth Rate of Total Population (%)	Growth Rate of Urban Population (%)
1960	93,419	40,830	43.7		
1965	98,275	47,261	48.1	1965/1960 1.02	1965/1960 2.97
1970	104,665	55,997	53.5	1970/1965 1.27	1970/1965 3.45
1975	110,940	63,823	57.0	1975/1970 1.35	1975/1970 2.65
1985	123,749	79,942	64.6	1985/1975 1.01	1985/1975 2.28
1990	128,272	86,327	67.3	1990/1985 0.72	1990/1985 1.55
2000	136,899	97,883	71.5	2000/1990 0.65	2000/1990 1.26

Table-7 Present and Future Demand for Water Resources in Japan

(billion cub. m./year)

Region	Demand of Water Resources by Region															
	1975								1985							
	Municipal	(%)	Industrial	(%)	Agricul-tural	(%)	Total	(%)	Municipal	(%)	Industrial	(%)	Agricul-tural	(%)	Total	(%)
Hokkaido	0.41	7	1.23	21	4.3	72	5.94	100	0.91	11	2.20	27	4.98	62	8.09	100
Tohoku	1.01	5	1.88	10	15.8	85	18.69	100	1.97	8	3.64	16	17.53	76	23.14	100
Kanto	4.34	25	3.39	19	10.0	56	17.73	100	7.50	31	5.07	21	11.49	48	24.06	100
Hokuriku	0.30	7	1.09	23	3.3	70	4.69	100	0.75	10	1.48	27	3.51	63	5.56	100
Tokai	1.52	16	3.72	40	4.1	44	9.34	100	2.55	19	5.95	45	4.75	36	13.25	100
Kinki	2.44	25	2.66	27	4.7	48	9.80	100	3.71	29	3.63	29	5.29	42	12.63	100
Chugoku	0.75	10	1.77	23	5.1	67	7.62	100	1.34	14	2.93	31	5.24	55	9.51	100
Shikoku	0.37	10	0.99	26	2.4	64	3.76	100	0.67	14	1.44	30	2.73	56	4.84	100
Kyushu	1.10	11	1.52	16	7.2	73	9.82	100	2.05	16	2.88	22	8.10	62	13.03	100
Okinawa	0.11	46	0.03	12	0.1	42	0.24	100	0.19	46	0.06	15	0.16	39	0.41	100
Whole Country	12.35	14	18.28	21	57.0	65	87.63	100	21.46	19	29.28	26	63.78	55	114.52	100

Table-8 Per Capita Water Supply by Population Size

Population (1,000 person)	Per Capita Water Supply ℓ/capita/d
Over 1,000	448
500 ∿ 1,000	371
250 ∿ 500	370
100 ∿ 250	350
50 ∿ 100	337
10 ∿ 50	304
5 ∿ 10	272
Under 5	197

Note: Data are for 1974

Under these circumstances the Government took steps to control groundwater by prohibiting the new construction of deep wells and the expansion of existing deep well in specified areas. They promoted the construction of public industrial water supply system. Water was supplied to industries in these specified areas at a comparatively cheap price. This was in compensation for the prohibition on groundwater usage. The cost of water for industries using public industrial water supply systems has been maintained at a low level by financial assistance from both central and local governments. Accordingly, these public industrial water supply systems expanded in recent years.

But recently it has been becoming increasingly difficult to secure new water rights for obtaining water from rivers, and more effective utilization and conservation have been developed at factories through the promotion of recycling. Reclaimed water from municipal effluents also has been used.

The water demand for agricultural uses is still increasing because the agricultural water supply and drainage network for paddy fields has expanded to facilitate the introduction of large, agricultural machinery. But requirements for agricultural water throughout country have changed recently, as shown in Table-7.

3. FUTURE DEMAND AND SUPPLY PROGRAM FOR WATER RESOURCES

The Ministry of Construction constructed a survey of "Supply Programs for Water Resources" in accordance with water demand by region, as shown in Table-9.

Most regional supply programs were based on constructing dams upstream in river basins. In recent years, it has become increasingly difficult to develop these plans, and dam construction has been delayed due to a decrease in appropriate dam sites and strong opposition against dam construction arising from local inhabitants. Consequently, water shortage areas, such as the large metropolitan areas of Tokyo, Kyoto, Osaka, Kobe and Northern Kyushu, have been forced to depend on unstable sources like the river basins of the Tone, Yodo and Chikugo rivers.

In order to cope with this situation, further efforts to encourage the public to conserve water, as well as the development of dam construction and water recycling are expected.

Recently there has been a growing recognition in Japan of the fact that water resources are limited just like oil and other mineral resources and it is very important to learn how to use the limited water resources and develop the technology for the recycling and reuse of wastewater. In this sense, we have to pay attention to the fact that effluents from municipal wastewater treatment plants will increase in accordance with the development of sewage collection systems. The construction of sewerage systems has been progressing smoothly. Of course, it will take time to develop further sewerage systems throughout the country, but it can be clearly understood that when 40%, 70% and 90% of the population are provided with sewerage systems, we shall have another 3.62, 11.83 and 193.2 billion cubic meters per year respectively from sewage effluents, as shown in Table-10. When we expect serious shortages of water, particularly in the dry season, we shall have to evaluate the value of municipal wastewater effluents and also recognize again the increasing importance in municipal wastewater quality control.

Table-9 Estimate of Water Demand and Supply in 1985 by Region

(billion cub. m/year)

Region		Additional Demand	Possible Supply	Balance
Hokkaido		1.91	2.27	0.36
Tohoku		4.14	4.46	0.32
Kanto	Northern Kanto	3.57	3.80	0.23
	Southern Kanto	5.20	4.51	Δ0.69
Hokuriku		1.07	1.24	0.17
Tokai		4.43	4.70	0.27
Kinki	Kyoto, Osaka, Kobe Area	1.97	1.66	Δ0.31
	Others	1.53	2.02	0.49
Chugoku	Japan Sea Side	0.33	0.38	0.05
	Seto Inland Sea Side	1.32	1.55	0.23
Shikoku		0.52	0.71	0.19
Kyushu	Northern Kyushu	1.02	0.80	Δ0.22
	Others	1.90	2.15	0.25
Okinawa		0.20	0.21	0.01
Whole Country		29.11	30.46	1.35

Table-10 Estimate of Municipal Effluents by Region in Accordance with the Development of Sewerage Systems

(billion cub. m/year)

Region		Population Sewered/National Population (%)			
		28	40	70	90
Hokkaido		0.28	0.47	0.79	1.26
Tohoku		0.26	0.41	1.38	2.51
Kanto	Northern Kanto	0.22	0.35	0.91	1.71
	Southern Kanto	2.22	3.65	6.10	7.40
Hokuriku		0.11	0.15	0.39	0.72
Tokai		0.73	1.03	1.91	2.86
Kinki	Kyoto, Osaka, Kobe Area	1.61	2.27	3.36	4.01
	Others	0.05	0.19	0.39	0.71
Chugoku	Japan Sea Side	0.02	0.06	0.15	0.26
	Seto Inland Sea Side	0.21	0.36	0.71	1.14
Shikoku		0.09	0.15	0.41	0.64
Kyushu	Northern Kyushu	0.22	0.43	0.78	1.15
	Others	0.11	0.19	0.60	0.96
Okinawa		0.03	0.08	0.12	0.16
Whole Country		6.17	9.79	18.00	25.49

Table-14 Present Status of Direct Reuse of Wastewater Effluents in Japan, 1978

Name of City	Name of Plant	Additional Treatment Process	Cub.m/d	Purpose
Hitachi	Ikenogawa	Rapid Sand Filter	30	Flush toilet
Takasaki	Jyonan		1800	Dilution for Night Soil Treatment Plant
Kawaguchi	Ryoke	Microstrainer	770	Washing water for Night Soil Treatment Plant
Funabashi	Nishiura		5900	Dilution for Night Soil Treatment Plant
Mohara	Kawanakajima		1000	"
Otsu	Otsu		500	Pilot Plant of AWT
Ashiya	Ashiya	Microstrainer	100	Spray Irrigation
Kakogawa	Ogami	Microstrainer	1200	Washing water for Night Soil Treatment Plant
Takasago	Takasago	Microstrainer	300	Industrial water use
Kurume	Tsufuku	Microstrainer	2600	Dilution water for Night Soil Treatment Plant
Iizaka	Iizaka	Rapid Sand Filter	550	"
Tokyo	Morigasaki	Rapid Sand Filter	980	Coolig & Washing of Solids Incineration Plant
Tokyo	Sunamachi	Coagulation & Rapid Sand Filter	3600	Industrial water
Tokyo	Shibaura		410	Washing Cars
Tokyo	Shibaura		145	Washing
Tokyo	Mikawajima		70310	Industrial water
Kawasaki	Iriezaki	Microstrainer	15417	Industrial water, cooling
Nagoya	Meijo		20	Pipe cleaning
Nagoya	Sennen	Coagulation & Rapid Sand Filter	33000	Industrial water, cooling, washing
Osaka	Nakahama	Rapid Sand Filter	3500	Landscape Irrigation
Kitakyushu	Hiagari		5464	Washing of Solids Incineration Plant
Kitakyushu	Kogasaki		15700	Industrial water
Fukuoka	Tobu	Coagulation, Rapid Sand Filter	1400	Industrial water

4. STRENGTHENING THE WATER QUALITY MANAGEMENT PROGRAM IN THE BASIN

Surface water development in the major river basins in Japan has been intense and additional dam sites for new development which are economical and physically feasible have been harder to find. Particularly during the dry summer when increased demand makes water levels harder to maintain in rivers, attention is now shifting to the low flow itself. Emphasis has been on controlling municipal and industrial pollution as the major causes of poor water quality during the summer low flows, but the quantity and quality relationship of surface water is not always adequately accounted for in present water resources plans.

The situation on this matter in Japan is as follows:

According to the institutional structure in Japan, water policy is administered by the National Land Agency. Water quantity is planned by the Ministry of Construction and several other Ministries are responsible for specified water utilization programs on agricultural water (the Ministry of Agriculture and Forestry), drinking water (the Ministry of Health and Welfare) and industrial water (the Ministry of International Trade and Industry).

Water quality is under jurisdiction of the Environmental Agency which is strongly oriented toward quality and sometimes is weak in dealing with quantity issues. Water quantity issues are more or less interrelated with water quality problems. Both quantity and quality in most rivers in urban areas are influenced by human activities and water quality may be at its poorest during the dry summer when streamflow is low and temperature is high.

Water supply and wastewater treatment projects are separately planned, financed and constructed at all levels, usually by different agencies. In a river basin indirect reuse through discharge of an effluent to a river and withdrawal downstream is a common practice and we should reemphasize the importance of interaction among wastewater management alternatives and area-wide water supplies.

From the viewpoint of water resources, the integrated planning and management of the quantity/quality of surface water should be organized. Coordinated management approaches between corresponding agencies for quantity/quality should be primarily considered, but as a result of intense surface water resource development and programs coordinated management approaches are generally hampered by many different agencies at the government level, each having its own different objectives for managing the same water resources. Each agency may often overlook key interrelationships which may result in mutual benefits.

The Comprehensive Basin Sewerage Plan of Sewerage Act 2-2 is intended to be a comprehensive study of all point pollution sources, leading to the development of a cost-effective plan to control these pollution sources in order to attain environmental water quality standards.

This study is to be coordinated between the Ministry of Construction and the Environmental Agency. The provisions of the Comprehensive Basin Sewerage Plan are; Sewerage Act Article 2-2 "Comprehensive Basin Sewerage Plan."

If rivers and other public bodies of water or coastal areas, to which the "environmental water quality standard" is applied to maintain a sound living environment in relation to water pollution as provided for Paragraph 1, Article 9 of the Basic Law for Environmental Pollution Control (P.L. 132 of 1967), satisfy the requirements as specified by the Cabinet Order, each prefecture shall present "a comprehensive basin sewerage plan" to the respective public bodies of water or coastal areas for the purpose of bringing the environmental conditions of the subject area in line with the "environmental water quality standard."

In the "comprehensive basin sewerage plan," each of the following items shall be specified clearly as stipulated by the Ordinance of the Ministry of Construction:

1. Basic policy on the improvement of sewerage
2. Matters related to the area where the effluent should be discharged and disposed of through a sewerage system

3. Matters related to the location, structure and functions of the basic facilities of sewerage systems in the area mentioned in the preceding paragraph.
4. Matters related to the sequence of execution of actual sewerage improvement works in the area specified by item 2.

The "comprehensive basin sewerage plan" shall be determined in consideration of each of the following matters;

1. Topography, precipitation of the specified area, flow of the river water and other natural conditions.
2. Prospective land use in the specified area.
3. Prospective water use in the subject water area for public use.
4. Prospective amount and quality of wastewater in the subject area.
5. Location where the wastewater is discharged.
6. Cost-effective analysis related to improvement of the sewerage systems.

In planning a "comprehensive basin sewerage plan" under the provisions of paragraph 1, the prefectural governments shall listen in advance to the opinions of prefectural and municipal officials concerned and shall obtain approval from the Minister of Construction as specified by the Ordinance of the Ministry of Construction.

The Minister of Construction shall consult with the Administrator of the Environmental Agency prior to giving the foregoing approval.

In the event that a revision has been made on the environmental water quality standard as provided for in paragraph 1, resulting in changes of matters specified under each item of paragraph 3, and other necessitating changes to be made in the "comprehensive basin sewerage plan," the prefectural authorities shall make the necessary changes on the "comprehensive basin sewerage plan" without delay. In this case, provisions from paragraph 2 to the preceding paragraph will be applied.

In the "comprehensive basin sewerage plan" coordination to various plans on water problems is absolutely necessary.

Consideration should be paid to expanding the scope of the "Comprehensive Basin Sewerage Study" by requiring this study to contain program areas which address:

1. Coordination of land use controls for nonpoint pollution sources
2. Water conservation (moderation of water demand) coordination
3. Recycling and reuse plan coordination
4. Agricultural irrigation water use plan coordination
5. Integrated quantity/quality program coordination
6. Low flow control plan and wastewater treatment plan coordination
7. Water supply and wastewater treatment plan coordination
8. Hazardous wastes disposal plan coordination

In these cases it is extremely important that hazardous wastes such as organic poisons and heavy metals be prevented from entering rivers and public sewers, because these materials might easily pass through conventional municipal wastewater treatment plants, and hazardous wastes would impair the environmental water quality standards. In any case, the "comprehensive basin sewerage plan" should help to act as a go-between for water quality and quantity plans.

5. CONSERVATION OF WATER

The term water conservation has been used so far to refer to supply rather than demand control, but it also means reducing the per capita demand on the municipal water supply. During the period from May 20, 1978 to March 24, 1979 Fukuoka City in Northern Kyushu experienced a ten month restriction in its water supply due to very severe drought. It was the worst drought in ninety years in Northern Kyushu. In light of the severe drought during this period, water conservation is very much on Fukuoka citizen's mind. Since the drought the inflow to the municipal wastewater treatment plants in Fukuoka City has been reduced as a result of a conservation movement by Fukuoka citizens and the development of a line of water saving fixtures. Conservation in the sense of demand control has been receiving increased recognition as a desirable alternative in solving water shortage problems.

Over the long term, conservation by Fukuoka City could reduce or delay the need to expand its water supply or wastewater treatment facilities. If the City of Fukuoka wants to obtain additional water rights to get more water, a large amount of funds would be needed to expand existing facilities. Thus, water conservation is an attractive policy for controlling water demand.

For the short term, however, in the absence of a phased program, as a result of conservation throughout Fukuoka City the Fukuoka Water Supply Department might have to raise rates to meet its financial obligations. Actually it is claimed by the Fukuoka Water Supply Department that annual revenue would decrease due to a water sales decrease and, consequently, water rate increases would outweigh savings from reduced water use. This is because public water supply systems in Japan, as well as the Water Supply Department of Fukuoka City, rely most frequently on the use of bonds to finance not only operating expenses but also capital costs by revenue from water sales. But we have to make another evaluation from a different angle of energy savings on municipal conservation.

There is another way to promote conservation which may be indicated in the case of City of Osaka. This case shows that when considering the conservation of water, an increasing and progressive sewer user charge system might be quite effective. In the pricing structure for water use and wastewater there is a trend toward either a fixed rate per cubic m. or an increasing and progressive rate per cubic m. in accordance with increasing demand. A pricing structure carefully developed and applied to the different classes of water use could be a fair and effective incentive for reducing excessive water use or wastewater. Of course, a pricing structure such as a rate system alone is not an adequate mechanism to conserve water. It may be that public education can help to form a concensus in conservation or be one of the incentives for encouraging more effective conservation and selecting those who should conserve.

In the case of the City of Osaka, the increasing and progressive sewer user charge system as shown in Table-11 was employed, and also the sewer user charge system according to strength shown in Table-12. As a result of application of these sewer user charge systems, the water usage pattern in the City of Osaka has changed significantly as shown in Table-13. The pollution load in terms of BOD or COD and SS from industrial dischargers into public sewers also has been dramatically reduced as shown in Fig. 2 and Fig. 3. It is clear from Table-13 that yearly flow from monthly dischargers of 10,001 cubic m. into public sewers have decreased year by year, but flow from dischargers less than 50 cubic m. per month, that is per average domestic sewage/family, have gradually increased year by year. From Fig. 2, it can be said that the higher BOD loading dischargers have been trying to reduce their BOD loads more effectively in order to pay less in sewer user charges. This may be one of the economic incentives for water conservation.

Table-11 The Change of Sewer User Charge System by Quantity in City of Osaka

User Discharge (Cubic m. per month)	1972 (yen)	1974 (yen)	1979 (yen)
Basic Rate up to 8 cub. m.	50	50	-
Basic Rate up to 10 cub. m.	70	70	100
Increasing Rate			
10 ∿ 20 cub. m. (per cub. m.)	10	10	20
21 ∿ 30 " "	15	15	27
31 ∿ 50 " "	16	16	30
51 ∿ 100 " "	17	17	35
Over 101 " "	18	-	-
101 ∿ 200 " "	18	20	40
201 ∿ 500 " "	18	25	50
501 ∿ 1,000 " "	18	30	60
Over 1,001 " "	-	-	-
1,001 ∿ 5,000 " "	18	40	80
Over 5,001 " "	18	40	90

Table-12 The Change of Sewer User Charge System by Strength (BOD or COD, SS)
- Surcharge for Industrial Users - in City of Osaka

(Unit: yen per cub. m.)

Strength (mg/ℓ)	1973		1974		1977	
	BOD (COD)	SS	BOD (COD)	SS	BOD (COD)	SS
201 ∿ 300	2	2	4	5	8	9
301 ∿ 450	6	7	13	16	18	21
451 ∿ 600	6	7	13	16	30	35
601 ∿ 850	13	16	28	36	46	55
851 ∿ 1,100	13	16	28	36	66	79
1,101 ∿ 1,350	23	27	50	61	86	103
1,351 ∿ 1,600	23	27	50	61	106	127
1,601 ∿ 1,850	33	38	72	86	126	152
1,851 ∿ 2,100	33	38	72	86	146	176
2,101 ∿ 2,350	42	49	91	110	166	200
2,351 ∿ 2,600	42	49	91	110	186	225

Table-13 Change of Sewage Discharge due to Increasing Sewer User Charge System in City of Osaka

User Discharge (cub. m. per month)	Sewage Discharge cubic meter per annum									
	1973		1974		1975		1976		1977	
	cub. m.	%	cub. m.	%	cub. m.	%	cub. m.	%	cub. m.	%
0 ~ 50	187,986	36.0	196,506	39.5	209,478	41.3	216,270	43.6	223,403	45.4
51 ~ 1,000	124,725	23.9	113,828	22.9	118,511	23.3	113,847	23.0	115,600	23.5
1,001 ~ 10,000	74,308	14.2	67,008	13.5	69,905	13.8	67,521	13.6	67,725	13.8
Over 10,001	135,612	25.9	120,177	24.1	109,493	21.6	98,121	19.8	85,192	17.3
Total	522,631	100.0	497,519	100.0	507,387	100.0	495,759	100.0	491,920	100.0

Fig. 2 The Impace of Sewer User Surcharge System by Strenth, BOD or COD in the City of Osaka

(Unit: ton/year)

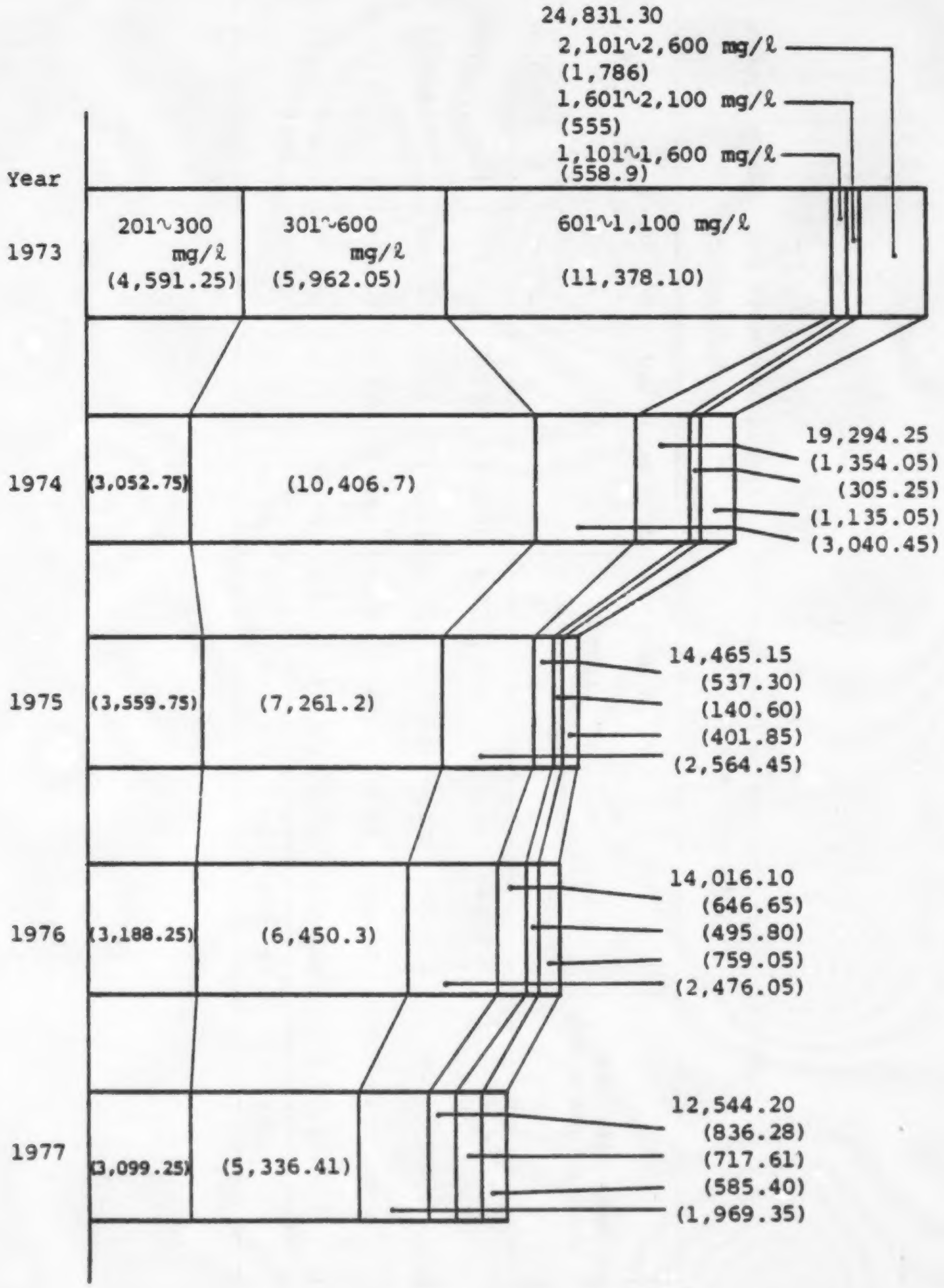
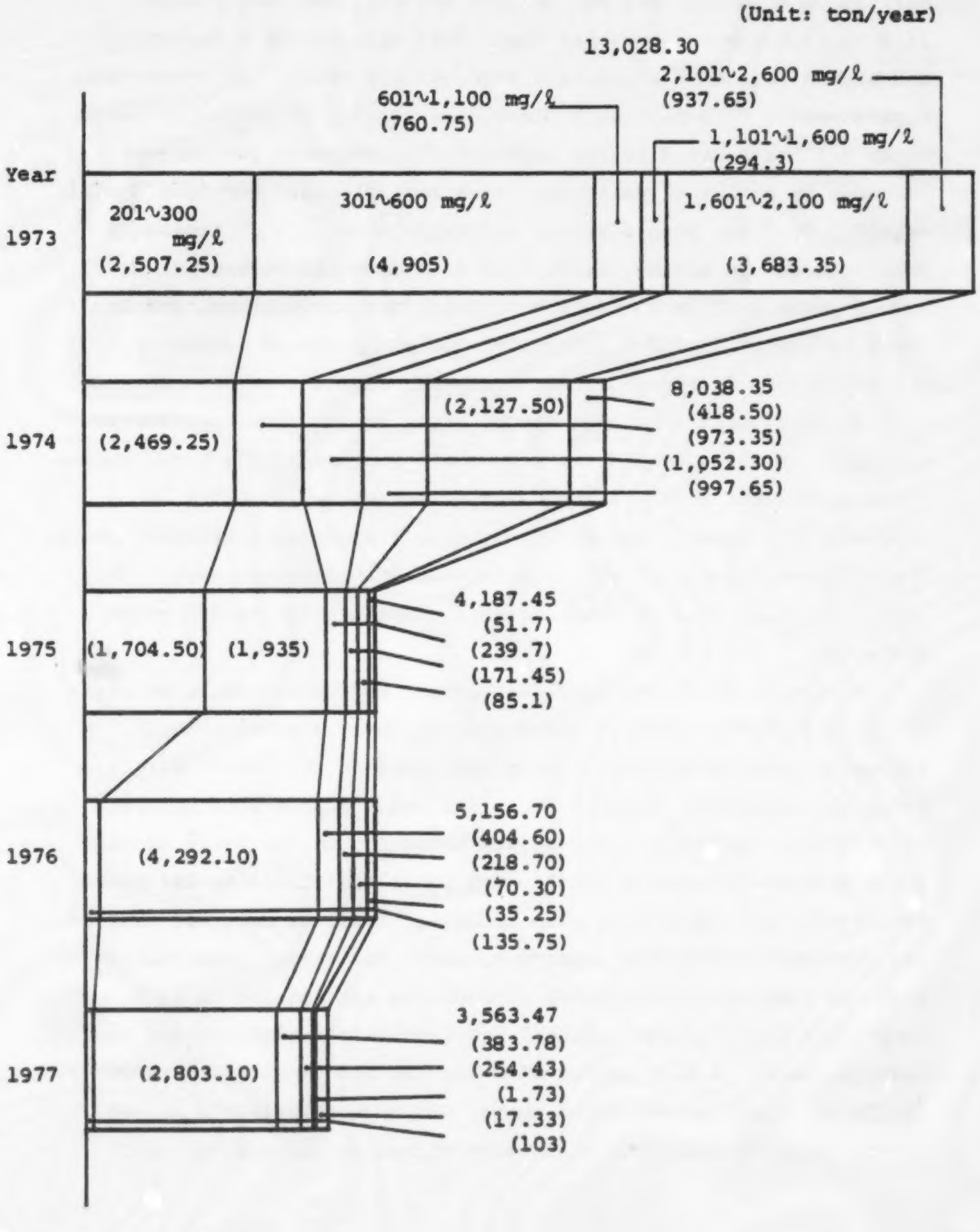


Fig. 3 The Impact of Sewer User Surcharge System by Strength, SS, in the City of Osaka



6. REUSE OF MUNICIPAL WASTEWATER EFFLUENTS

6.1 Present Status of Municipal Wastewater Effluents Reuse

Total municipal wastewater discharge in Japan amounted to 6.17 billion cubic meters per day in 1978 and will eventually amount to 25.5 billion cubic meters per day. This will become a dependable and stable source of relatively high-quality water. In comparison, approximately 0.73 million cubic meters per day of municipal wastewater effluents are directly reused in 201 projects for various purposes by municipal wastewater treatment plants. Purposes include washing, cooling, pump sealing, defoaming etc. through treatment processes at the sites. About 0.16 million cubic meters per day are directly used in 22 projects, mostly for agricultural and landscape irrigation and for industrial cooling purposes primarily in the water-scarce areas shown in Table-14.

These figures show that direct reuse of municipal wastewater effluents at this moment is a very small portion of the total municipal wastewater discharge in Japan. Each of the examples in Table-14 is a classic example of municipal wastewater effluent reuse. These reuse projects have been conducted as pioneering work. Thus, their main significance might be as a precedent for future reuse projects.

Direct reuse of municipal wastewater effluents should be carefully considered. This involves a pipeline or similar conduit, frequently including pumping from the plant to the user. Direct reuse is limited to specific locations and requires site analysis of technical, economic alternatives according to the local conditions and the importance of allowing local conditions to determine the system most suited for each situation. Direct reuse of municipal wastewater effluents appears to offer additional potential to ease the competition for water through its application to such nonpotable users as agricultural and landscape irrigation and industrial uses. A huge amount of water has been applied to paddy fields in Japan, as mentioned above, and some experiments in the direct reuse of municipal wastewater effluents for agricultural irrigation to paddy fields have been going on in several sites paying particular attention to nitrogen content in the effluents. Water reuse for agricultural irrigation would be reasonably attractive and economical in some parts of Japan.

In promoting the reuse of municipal wastewater effluents, viruses and new toxic chemicals require additional research even before they can be indirectly reused in municipal wastewater effluents. Direct reuse of municipal wastewater effluents for the drinking water supply cannot be considered at this moment in Japan. Further research and experimentation are absolutely requried to provide additional assurance on the safety and health effects. But direct reuse of municipal wastewater effluents can provide a mechanism for obtaining drinking water supplies from other uses by exchanging and substituting treated reclaimed municipal effluents. This means that the exchange of water rights among different water right owners through a mechanism of reuse systems would be expected.

Reuse means here use of municipal wastewater effluents for some nonpotable purposes. Such reuse is already widespread for indirect reuse, wherein municipal wastewater discharged to a river by one user is withdrawn downstream by another. Thus, downstream water-users such as water supply, agricultural and industrial users are quite often involved in the water usage cycle in a river basin and wastewater may be put through several cycles of indirect reuse in the river basin. The Water Supply Department of Kyoto City has been drawing water for drinking purposes from Lake Biwa, located in the Yodo River Basin. After usage, Kyoto's wastewater effluents have been discharged downstream in the same Yodo River. The major water source of the Tokyo Metropolitan Water Supply Department has been taken from the Tone River. Almost all local municipalities in Gunma Prefecture are located upstream on the Tone River. Their wastewater has been discharged directly into Tone River and withdrawn downstream for drinking purposes in the Tokyo Metropolitan area. Indirect reuse of municipal wastewater effluents through discharge of an effluent to a river and withdrawal downstream, is recognized to be so important that the complexities of reuse also need to be recognized and evaluated.

6.2 Problems to Be Examined and Studied in Direct Resue of Municipal Wastewater Effluents

1. Water Quality Standard for Direct Reuse

There is no water quality standards for direct reuse of municipal wastewater effluents in Japan. There is still significant potential uncertainty relating to the health effect associated with some types of direct reuse of municipal wastewater effluents. Examples include the irrigation of food crops, contamination of groundwater sources used for drinking water and recreation, and the degree of treatment needed to avoid health risks. Advanced waste treatment and disinfection may be desirable for these purposes, but it appears that at least secondary treatment and disinfection may be required for direct reuse of municipal wastewater effluents for these purposes. This is mainly due to economic considerations.

2. The Economics of Direct Reuse of Municipal Wastewater Effluents

The economics of direct reuse of municipal wastewater effluents is a major factor which will limit the extent of its implementation. Foremost is cost of reuse in relation to the cost of other sources of supplies and alternatives to additional supply, as well as the cost and high energy use of the technology itself in the case of advanced waste treatment. Both capital and operational expenditures for direct reuse of municipal wastewater effluents tend to be high. Transportation and storage of reclaimed effluents are usually required. Pipelines often containe pumping systems wihich may increase transportation costs. The economics associated with advanced waste treatment processes include the high cost of construction and operation, large requirements for energy and chemicals, and the increased amount of sludge produced in advanced waste treatment processes which require disposal.

If we consider the competition from alternative sources, existing municipal water supplies and public industrial water supplies in Japan can often be provided at less expense than would be required for direct reuse of municipal wastewater effluents. This is because existing municipal water supplies and public industrial water supplies usually come from relatively open sources which were developed at pre-inflation prices many years ago. Public industrial water supplies are financed by a grant and loan system in compensation for the prohibition of groundwater pumping as mentioned above.

One of the problems for direct reuse of municipal wastewater effluents is the primary market for municipal wastewater effluents. Reuse for agricultural irrigation, which may be quite attractive, is often found among users least willing to pay for water. Sometimes such water is free for historical reasons. When a reuse project is analyzed, its desirability is strongly affected by the magnitude and source of financial assistance, such as grants and loans from federal government. Thus water economy problems should be reconsidered more carefully.

3. Energy Consumption in the Direct Reuse of Municipal Wastewater Effluents

The potential need for treatment and transportation may make municipal wastewater effluent reuse projects relatively energy consumptive. Energy implications must be evaluated and technological developments to save energy are also needed in this field. Energy problems in Japan will become more severe in the future.

4. Water Rights

The right to use water has certain attributes of the property right. Water rights for river water should be obtained by permission of each corresponding river authority. This is determined by River Law, but many water rights concerned with existing agricultural irrigation are sometimes uncertain and historically prescriptive. Reuse is possible if it allows increased riverflow and/or reduced discharges and hence reduced river flow but improved river water quality. Reuse, therefore, can provide a mechanism for adjusting water rights among various water right owners by exchanging or substituting treated reclaimed effluents. This mechanism will be explored in more detail in the future.

5. Psychological Problems Resulting from the Direct Reuse of Municipal Wastewater Effluents

There are many examples which involve the psychological inability to accept the direct reuse of municipal wastewater effluents due to lack of scientific consensus on the degree of treatment needed to protect against risks to health.

Public educations and the acceptance of municipal wastewater effluents in relation to reuse are sometimes emotional and volatile issues, but the importance of these psychological problems must not be neglected and public education on the scientific aspects of water reuse may be necessary to overcome them.

UNITED STATES PAPERS

MUNICIPAL TREATMENT REQUIREMENTS AND PRACTICES TO MAINTAIN WATER QUALITY IN THE TAMPA AND ESCAMBIA BAY AREAS.............. 365
Howard L. Rhodes, Division of Environmental Programs, Florida Department of Environmental Regulation, Tallahassee

INSTRUMENTATION AND AUTOMATION CONTROL OF MUNICIPAL SLUDGE TREATMENT FACILITIES.. 453
Irwin J. Kugelman, Municipal Environmental Research Laboratory, ORD, USEPA, Robert C. Polta, Ph.D., and Don Stulc, Metropolitan Waste Control Commission, and George A. Mathes, EMA, Inc., St. Paul, Minnesota

IMPACT OF INNOVATIVE AND ALTERNATIVE TECHNOLOGY IN THE UNITED STATES IN THE 1980's... 515
John M. Smith and Jeremiah J. McCarthy, Municipal Environmental Research Laboratory, ORD, USEPA and Henry L. Longest II, Office of Water Programs, ORD, USEPA

PARALLEL EVALUATION OF BELT FILTER PRESSES AND LOW SPEED SCROLL CENTRIFUGES.. 579
Walter E. Garrison and Robert W. Horvath, Los Angeles County Sanitation Districts, Whittier, California

THERMAL CONVERSION OF SLUDGE IN A MULTIPLE HEARTH FURNACE, USING A SUB-STIOCHIOMETRIC SUPPLY OF OXYGEN.................... 607
Joseph B. Farrell, Municipal Environmental Research Laboratory, ORD, USEPA

OCCURRENCE AND REMOVAL OF TOXICS IN MUNICIPAL WASTEWATER TREATMENT FACILITIES.. 633
John J. Convery, Jesse M. Cohen and Dolloff F. Bishop, Municipal Environmental Research Laboratory, ORD, USEPA

HEALTH EFFECTS RESEARCH ASSOCIATED WITH MUNICIPAL WASTEWATER TREATMENT AND SLUDGE DISPOSAL................................. 707
Herbert R. Pahren, Health Effects Research Laboratory, ORD, USEPA

MUNICIPAL TREATMENT REQUIREMENTS AND PRACTICES TO MAINTAIN WATER QUALITY IN THE TAMPA AND ESCAMBIA BAY AREAS

by

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ABSTRACT

This paper was written with the objective of showing the relationship of regulation of sewage works to the maintenance or improvement of water quality in the receiving body of water. Two Florida estuaries were examined in this report, Tampa and Escambia Bays.

Reviews of past data and regulatory schemes that led to upgrading sewage treatment works were made for comparison purposes with the state of the environment today.

These reviews led the author to conclude that as important as sewage discharges are to water quality in estuaries, problems were caused by other activities of man that had as much, if not more, impact on water quality. These activities included industrial discharges, dredge and fill operations, and non-point source discharges. Each appears to have a synergistic effect on water quality in estuaries. Thus, each will have to be addressed to solve the water quality problems of the past.

Based on data today, Escambia Bay is very slowly recovering from pollution of the 1960's. Tampa Bay is also showing marginal signs of improvement. Regulatory action in the municipal, industrial, drege and fill and non-point source sectors have together begun to positively impact water quality in these systems.

ACKNOWLEDGMENTS

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SECTION 1

INTRODUCTION

In the mid 1960's, the State of Florida encountered numerous instances of environmental problems including fish kills, eutrophication, accelerating growth of water weeds, turbidity, bacterial increases and loss of aquatic habitat. In response to the increased environmental awareness by the public of the State, the Florida Legislature created the Florida Air and Water Pollution Control Commission.

This Commission saw the solution to the pollution problem as being the control and containment of point source discharges both municipal and industrial. This strategy was followed for the two years the Commission was in existence. The successor agency was the Department of Pollution Control in 1969. This agency existed until a reorganization occurred in 1975 combining it into the Florida Department of Environmental Regulation (FDER).

These agencies were mandated by the State Legislature to require secondary treatment of sewage and industrial waste by January 1, 1973. This was defined by an administrative rule to be 90 percent removal of pollutants in 1969. The defacto policy of the Commission and the Departments was to bring about the treatment of wastewater as expeditiously as possible. Implementation of this policy did not in fact allow for a cause-effect relationship to be established between discharges and water quality. Monitoring that would have shown the impact of this policy was not implemented. Raw or primary discharges were to be cleaned up as a matter of public policy as soon as possible. Later, in 1972, federal law and policies reinforced this policy and little cause/effect was demonstrated nor improvements monitored.

For industry, this policy led to voluntary compliance with administrative orders, consent orders and court orders. Most of the industry was very compliant and worked to achieve the secondary standards in 1972 and 1973. By the end of 1973, most industries had attempted to comply with the secondary standard.

Municipal compliance was another issue, however. The U.S. Congress was debating the Water Pollution Control Act during 1971-1972 until its final enactment in October 1972. Provisions were being made to provide for construction grants for municipal facilities somewhere in the range of 50 and 80 percent of construction cost. A figure of 75 percent was finally selected. For a three year period, there was a virtual standstill in new construction that would solve municipal point source problems because municipalities waited for federal grant dollars. In retrospect, it could be reasonably argued that more would have been accomplished if no grants had been available. This issue is debatable though and it might be true that without the grants less would have been achieved because every regulatory decision would have been more closely scrutinized. At any rate, using this grant program, nearly all 1980 Florida dischargers will be capable of providing secondary treatment or better.

Sewage treatment was subjected to special requirements in two areas of the State. Two separate areas of the State had special water quality problems and special events happened that pushed these two areas of the State toward treatment above secondary into advanced treatment for surface water dischargers.

Responding to constituency desires, numerous water quality problems and a common belief that secondary treatment was inadequate in the Tampa Bay system and other bays of west-central Florida, two State Legislators decided to file a regional bill to require a minimum treatment for municipal waste of the highest order, i.e. Advanced Waste Treatment (AWT). AWT was later defined by administrative regulation to be effluent standards of:

Biochemical Oxygen Demand (BOD) - 5 mg/l
Suspended Solids (SS) - 5 mg/l
Total Nitrogen (TN) - 3 mg/l
Total Phosphorus (TP) - 1 mg/l

This regional bill defined a specific geographic area. This geographic area's population was thus required to pay more for sewage treatment than other people in the State.

This bill's passage was not so much that water quality was bad, since that fact was well recognized, but another ramification. The bill's passage was caused by the environmental political atmosphere of the times, by a strong crusade of environmental organizations in the Tampa Bay area and by pleas from a local Girl Scout group. This Girl Scout organization did not want secondarily treated sewage discharging into the ocean passing 3-8 kilometers from the camp. Emotionalism and the spirit of the times made a bill of this nature hard to oppose. Thus, with the passage of a single bill, AWT was required for all bays in west-central Florida.

Water quality studies done before this law were only marginally considered in its adoption. Water quality problems existed before, during and after the law's passage. Conseeffect relationships were speculated on but they did not form the basis of the statute.

In retrospect, there is doubt that water quality would ever have improved if a cause-effect relationship had to first be proven. The fast growing area would most likely have only further damaged the water quality with secondary treatment. The estuary systems are far too complex to solve with simple answers. Even today the exact cause of some of the water quality problems are not fully understood. As proven in the United States for a period of forty years, water quality standards alone did not control pollution. Regulatory practices in the United States were targeted in the 1940's, 50's and 60's to cause-effect water quality solutions. They did not work. It took a uniform technology standard to clean up many waterways. This does not imply though that water quality studies cannot be used to refine the degree of treatment or the pollutants removed but these studies are not exact sciences and the legal profession can create numerous roadblocks to effective regulation in such an atmosphere.

The other special requirement came about as a result of massive fish kills in Escambia Bay in 1969-1971. The Governor of Florida in August 1969 demonstrated to the citizens that he was doing something about the problem by calling on the Federal Water Quality Administration to convene a Federal-State Enforcement Conference. The purpose of such a conference was for state and federal enforcement officials to ascertain the cause of the fish kills as best they could and recommend corrective actions. The recommendations were for all industries and the municipality to provide very high levels of treatment prior to discharge into the Escambia Bay. Also, further dredging was prohibited and a physical obstruction (an abandoned railroad tressel) was recommended to be removed.

The industries complied with the subsequent consent orders, the tressel was removed and the dredging stopped; however, the municipality was caught up in an array of problems, including passage of the 1972 Water Quality Law. At first the City of Pensacola was to provide this same high level of treatment to its Northeast Sewage Treatment Plant (STP), as was required by the industries. Later, the City proposed connecting to the Pensacola Main Street Plant. On March 2, 1973, the U.S. Environmental Protection Agency (EPA) wrote a letter containing the following excerpt to the area regional planning council.

> "2. Determining the impact of waste
> discharges--specifically that of the
> Pensacola Main St. Plant--was addressed
> by the Technical Advisory Committee.
> The professionals listed in the minutes
> of those meetings acknowledged the
> limitations of mass balance calculations

> such as the ones performed by both project consultants and EPA. Based on available data and their judgment, it was agreed that changing water quality had degraded biological and aquatic productivity in the Bay system and there was sufficient evidence of a sensitivity to nutrients. A simplified dissolved oxygen analysis therefore was concluded to be incapable of providing the full range of considerations relevant to water quality enhancement in Pensacola Bay. It was recognized that expansion of a 9 MGD [$3.4X10^4$ m^3/day] facility to 18 MGD [$6.8X10^4$ m^3/day] while maintaining 90 percent BOD_5 removal would ultimately double wasteloads to the Bay. Clearly, neither maintenance nor enhancement could result from an overall increase in wasteloads. Within the context of enhancement and non-degradation, the TAC concluded that no additional wasteloads should be discharged from the Pensacola Main St. Plant. Any expansion, therefore, would require higher degrees of treatment commensurate to the load constraints of the present discharge."

Several water quality studies were made before and after these decisions. An exhaustive three year study was made by EPA shortly after the above letter was written and it did not show any significant new findings. There continued to be support for the decision not to increase the mass loadings to the Bay. Thus, the future expansion had to be a higher degree of treatment than secondary. The City of Pensacola then proposed AWT of BOD-5 ppm, SS-5 ppm, TN-3 ppm and TP-1 ppm. This proposal was accepted by the regulatory agencies and a $7.56X10^4$ m^3 day AWT Plant ultimately constructed. Cooperation between the City and the regulatory agencies was good until seven years later when the City protested the level of treatment. To this day, the issue is not fully resolved to everyone's satisfaction.

Thus, with both of these special requirements came mandatory decisions to provide AWT for municipal dischargers into the Escambia and Tampa Bay systems. The Tampa Bay system has had extensive modelling in subsequent years, but it still does not tell what cause-effect relationships exist. Modelling of the Escambia Bay system has not been done and there are questions whether it can be economically done with the present state of the art techniques. Conclusions regarding the effectiveness of these arbitrary pollution methods will be explored later. In the following pages a comparison is made between past conditions, what has happened and what is the present status of municipal treatment and water quality in Escambia and Tampa Bays.

SECTION 2

ESCAMBIA BAY

PHYSICAL CHARACTERISTICS AND HISTORY

The Escambia Bay system is comprised of three bays, Escambia Bay, Pensacola Bay and East Bay. These Bays are influenced by the Gulf of Mexico and Santa Rosa Sound. Escambia Bay is the system most impacted by pollution problems. This area's average temperature is 20°C and receives an annual rainfall of 152 centimeters per year. The surrounding topography ranges from gentle sea level slopes to heights of 33 meters above sea level.

Escambia Bay ranges in depth from .3-7 meters, averaging 2.5 meters at mean low water. The surface area of the Bay is 10,000 hectares with a 33 meter wide navigation channel traversing the Bay north to south at a depth of ten feet. The maximum freshwater flow of the Escambia River is 2.09×10^3 m^3/sec and the minimum is 1.61×10^1 m^3/sec.

Tidal effects consist of one high tide and one low tide daily with a tidal range of .5 meters. 18.8 percent of the Bay volume is potentially exchanged every tidal cycle due only to tidal effects. The displacement time for freshwater to replace the full volume of the Bay is 18 days and 3.2 days depending on flow. Assuming complete mixing, 5.3 days would be required to completely flush the Bay volume by tidal exchange alone. Physical displacement time at low river flow is 92 days, at mean annual discharge 18 days and high river discharge 1.6 days.

The northeast section of the Bay has poor tidal exchange and the discharge of the Escambia River discharging to the southeast hinders total exchange of the saline water flowing northward in the upper Bay. Thus, a short-circuiting effect occurs in the northeast section of the Bay where the industrial discharges occur. (See the following four graphs for current patterns at high and low flows: Figures 1, 2, 3, 4.)

Over the years, Escambia Bay has suffered an estimated 80 percent decrease in fishing value since 1952. The Bay suffered 20 fish kills in 1969, 75 fish kills in 1970, approximately 70 fish kills in 1971 and a massive oyster kill in 1971. (See Table 1 for relative size of fish kills.) It was shown tha disease caused the kills; however, there is reason to belie that the oysters had been stressed by pollution, dredging and

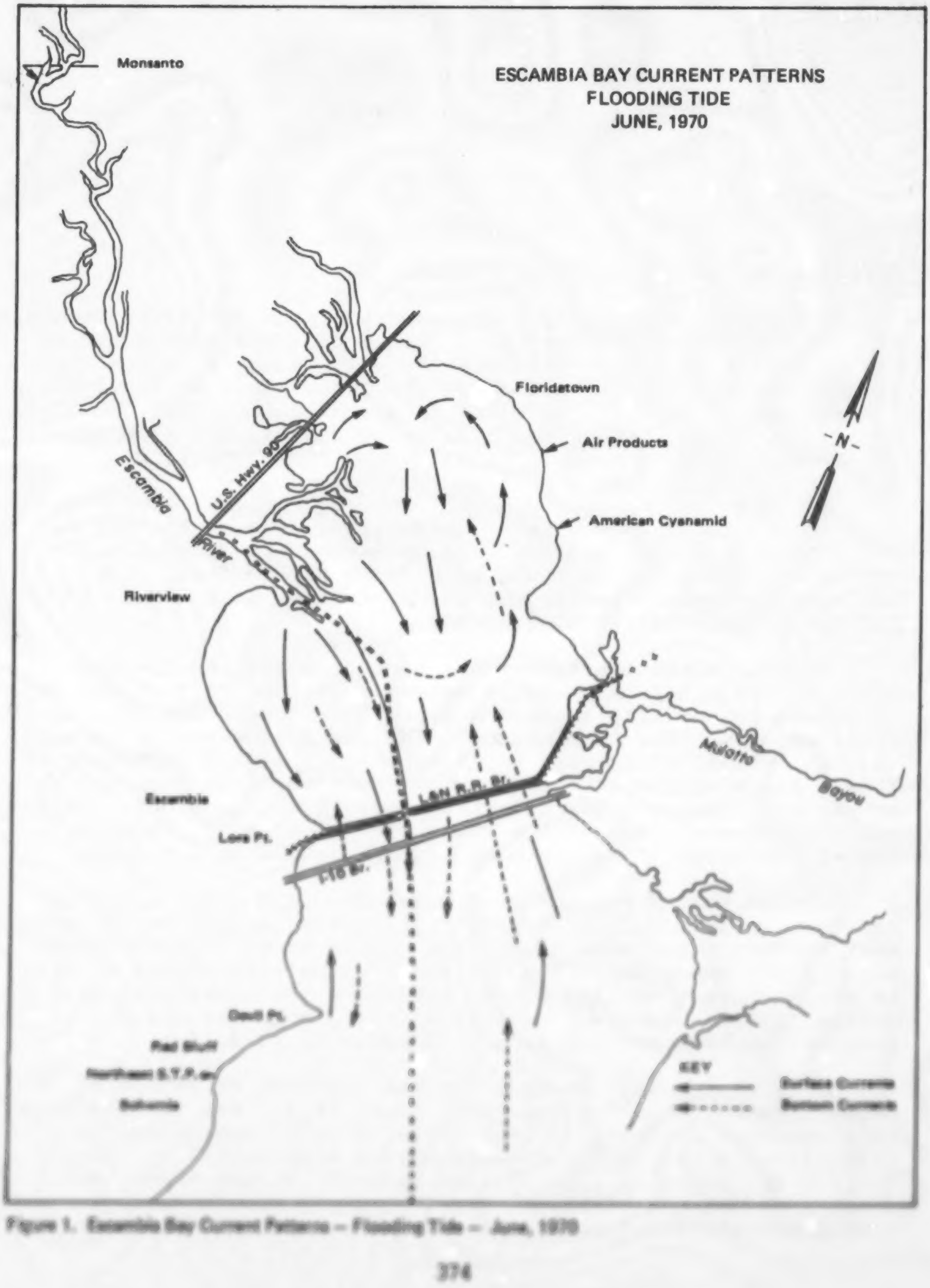
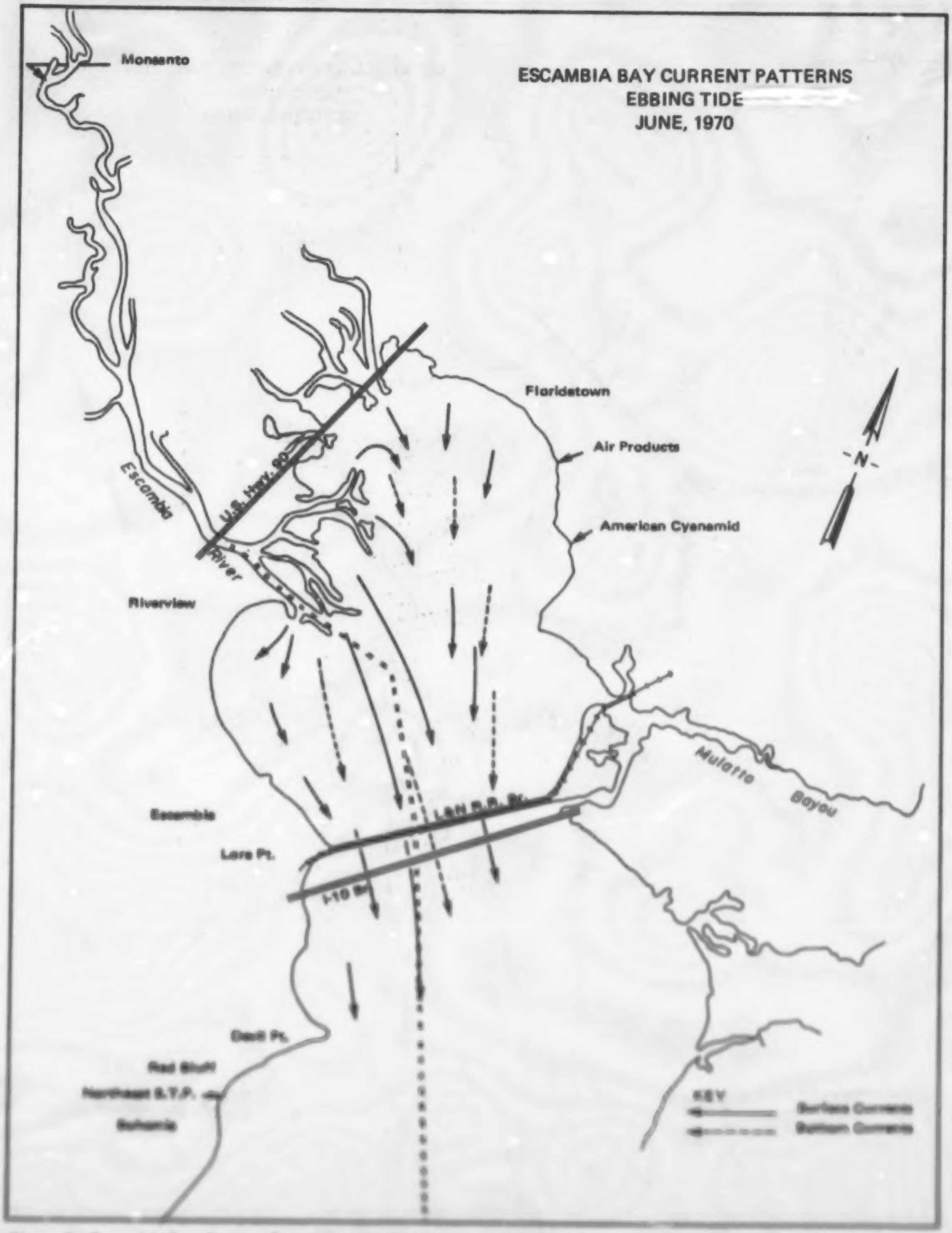

ESCAMBIA BAY CURRENT PATTERNS
FLOODING TIDE
JUNE, 1970
Monsanto
Floridatown
Air Products
American Cyanamid
N
U.S. Hwy. 90
Escambia River
Riverview
Mulatto Bayou
L&N R.R. Br.
Escambia
Lora Pt.
I-10 Br.
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
KEY
Surface Currents
Bottom Currents

Figure 1. Escambia Bay Current Patterns – Flooding Tide – June, 1970


ESCAMBIA BAY CURRENT PATTERNS
EBBING TIDE
JUNE, 1970
Monsanto
Escambia
River
U.S. Hwy. 90
Floridatown
Air Products
American Cyanamid
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Riverview
Mulatto
Bayou
L&N R.R. Br.
I-10 Br.
Escambia
Lora Pt.
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
KEY
Surface Currents
Bottom Currents

Figure 2. Escambia Bay Current Patterns – Ebbing Tide – June, 1970

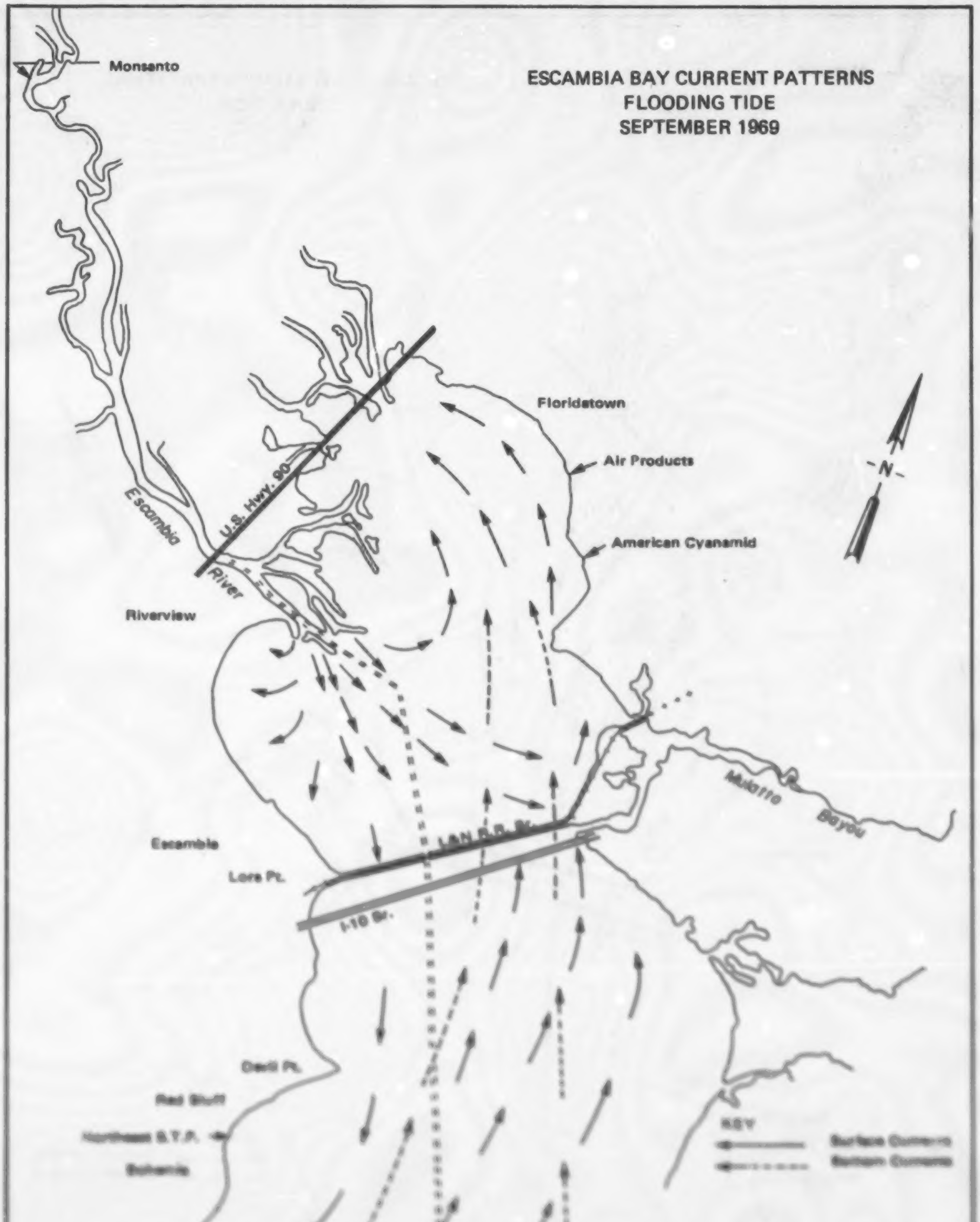
ESCAMBIA BAY CURRENT PATTERNS
FLOODING TIDE
SEPTEMBER 1969
Monsanto
Floridatown
Air Products
American Cyanamid
N
U.S. Hwy. 90
Escambia River
Riverview
Mulatto Bayou
L&N R.R. Br.
I-10 Br.
Escambia
Lora Pt.
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
KEY
Surface Currents
Bottom Currents

Figure 3. Escambia Bay Current Patterns — Flooding Tide — September, 1969

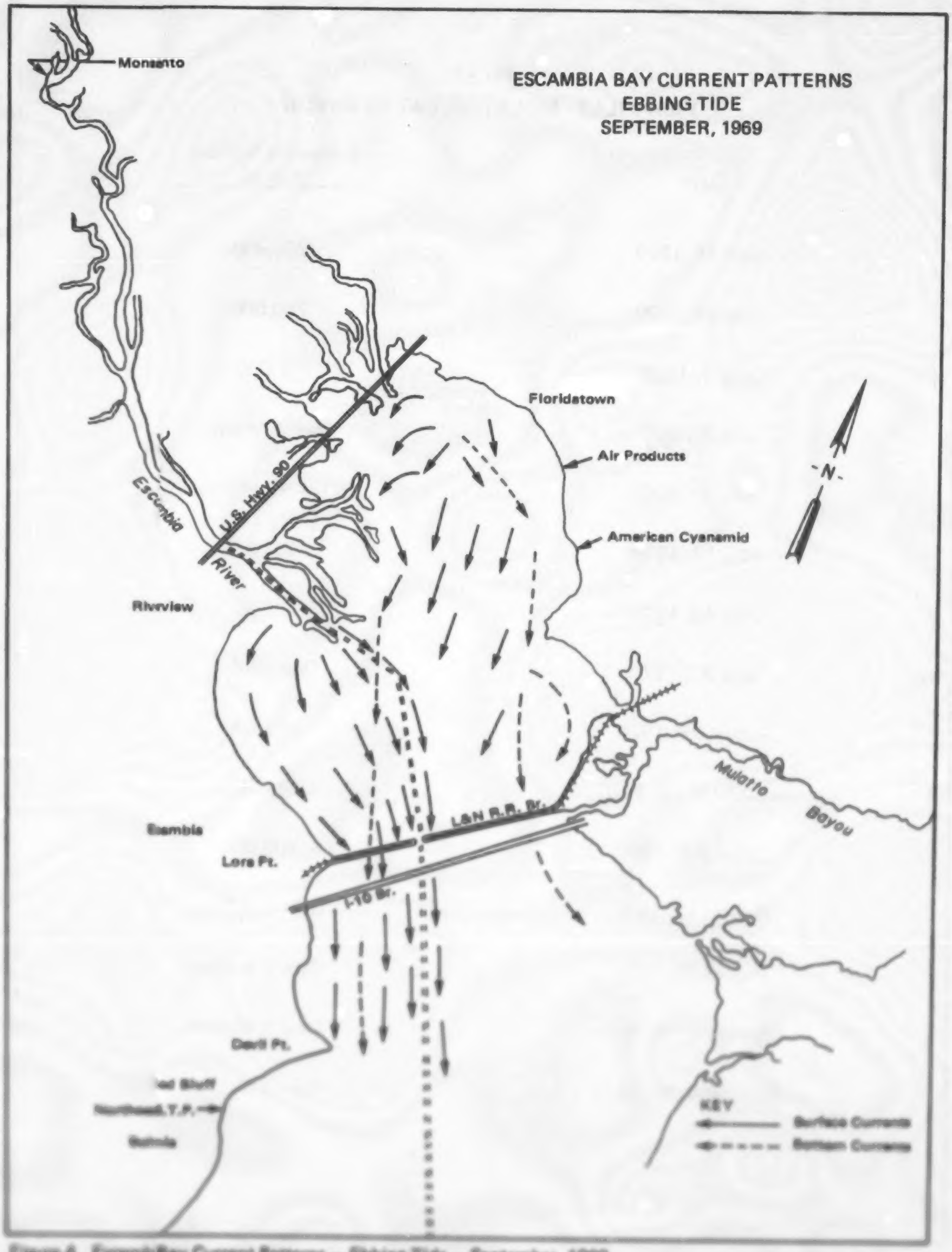
ESCAMBIA BAY CURRENT PATTERNS
EBBING TIDE
SEPTEMBER, 1969
Monsanto
Floridatown
Air Products
American Cyanamid
N
U.S. Hwy. 90
Escambia
River
Riverview
Mulatto
Bayou
L&N R.R. Br.
Escambia
Lora Pt.
I-10 Br.
Devil Pt.
Red Bluff
Northeast T.P.
Bohemia
KEY
Surface Currents
Bottom Currents

Figure 4. Escambia Bay Current Patterns – Ebbing Tide – September, 1969

TABLE 1.

FISH KILLS - MULAT/MULATTO BAYOU

Date	Estimated Number Killed
June 21, 1970	250,000
June 29, 1970	750,000
July 1, 1970	2,000
July 5, 1970	Over 1 Million
July 7, 1970	Over 10 Million
July 12, 1970	10,000
July 13, 1970	2,500
July 27, 1970	One Million
August 25, 1970	500-700
September 28, 1970	One Million
October 8, 1970	200,000
August 12, 1971	Over 1 Million
August 22, 1971	Over 2 Million
September 12, 1971	Over 1 Million
September 15, 1971	Over 3 Million

Polychlorinated Biphenol (PCB) contamination and were most susceptible to disease. In 1971, virtually every organism taken from the water had PCB residues. Sediments even today contain PCB residue. Marine grasses were reduced by roughly 70 percent, apparently by pollution, dredging and filling and shoreline alterations. Shrimp production decreased during the period.

A fish camp operator stated in 1970 that he bought $500 worth of live shrimp for use as bait for speckled trout, red fish and flounder per year. Five to seven years previously he was purchasing $10,000 worth of shrimp per year. His boat rental records show similar declines.

The Federal-State Enforcement Conference occurred in late 1969 with several conclusions, the most important of which were: (1) point source discharges were to remove 94 percent of the carbonaceous material, 94 percent of the nitrogenous material and 90 percent of the phosphorus, and virtually all settleable solids: (2) no further dredging in the Bay until a plan was established to handle dredged material on upland disposal sites; and (3) the abandoned railroad tressel was to be removed.

The relative contribution of industry and the Northeast STP for each constituent is shown in Figures 5, 6 and 7. As can be seen, BOD was more a function of industry and was relatively small for sewage. However, with regard to phosphorus and Total Kjeldahl Nitrogen (TKN), the sewage treatment plant made major contributions. Table 2 shows the final loads allowable under the Enforcement Conference recommendation.

These recommendations were based primarily on levels of carbonaceous, phosphorus and nitrogen that appeared to cause problems in similar waters or in experimental work. Thus, levels of unstabilized carbon of 0.26 mg/l, nitrate nitrogen of 0.03 mg/l and 0.01 mg/l were set as targets based on unpolluted waters to bring pollution in Escambia Bay under control.

Industry, under consent orders with the State of Florida, began to clean up their discharges. Some regulatory action has continued but initial efforts were satisfying.

The Northeast STP had similar pollutant removal requirements as those recommended for industry. The City has chosen to combine sewage plants and provide AWT. Likewise, the City decided to help clean up interior tributaries flowing through the City to the Bay because of tributary pollution problems caused by package plants within the City. These smaller STP flows were transmitted to the main plant via interceptor trunk lines, thus moving all sewage discharges to a single point. Federal grant dollars were used to centralize the discharges and to build the AWT Plant. This single AWT Main Street Plant is

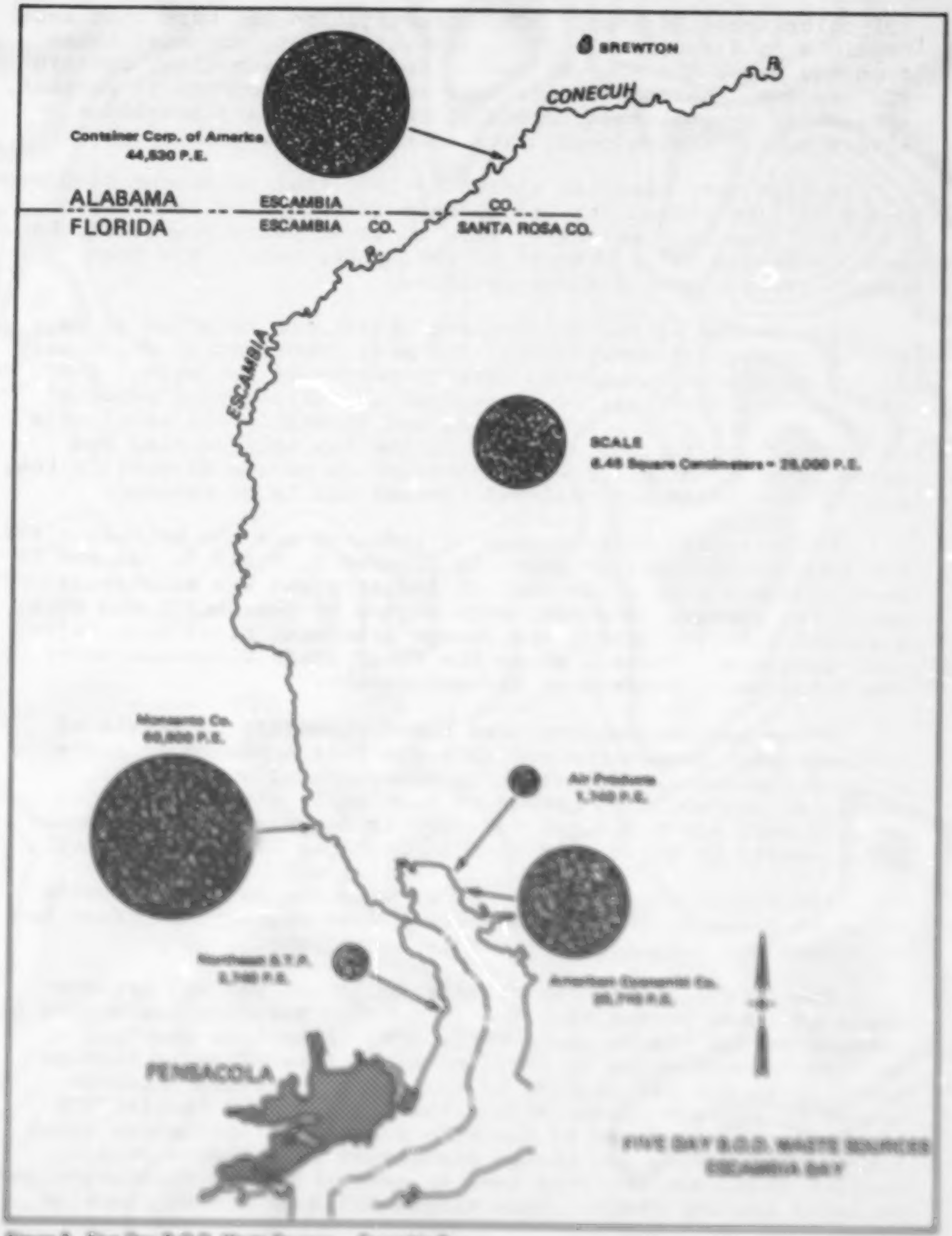
BREWTON
CONECUH R.
Container Corp. of America
44,530 P.E.
ALABAMA
FLORIDA
ESCAMBIA CO.
ESCAMBIA CO.
SANTA ROSA CO.
ESCAMBIA R.
SCALE
6.45 Square Centimeters = 25,000 P.E.
Monsanto Co.
60,000 P.E.
Air Products
1,740 P.E.
Northeast S.T.P.
2,740 P.E.
American Cyanamid Co.
20,710 P.E.
PENSACOLA
FIVE DAY B.O.D. WASTE SOURCES
ESCAMBIA BAY

Figure 8. Five Day B.O.D. Waste Sources – Escambia Bay

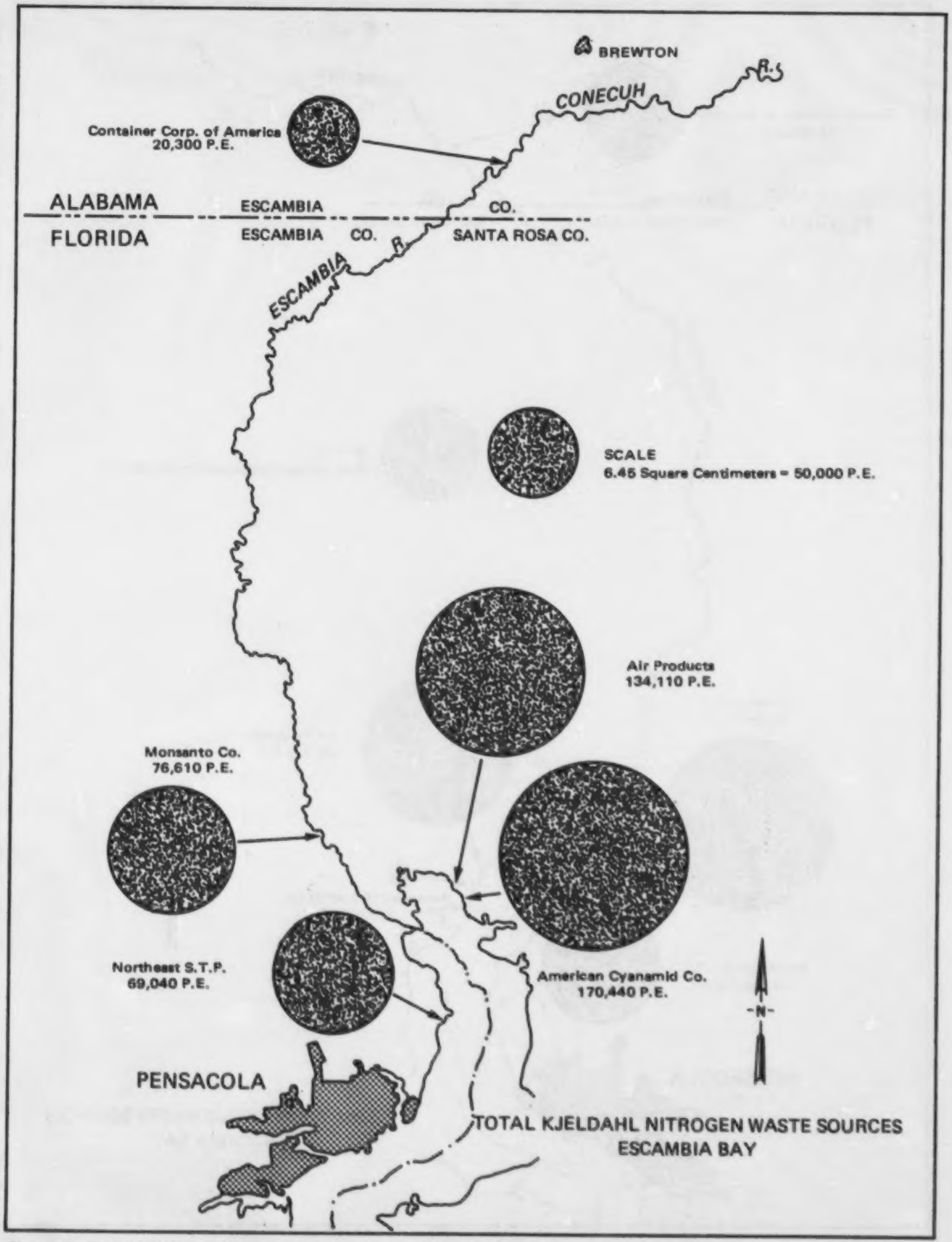
BREWTON
CONECUH R.
Container Corp. of America
20,300 P.E.
ALABAMA
ESCAMBIA CO.
FLORIDA
ESCAMBIA CO.
SANTA ROSA CO.
ESCAMBIA R.
SCALE
6.45 Square Centimeters = 50,000 P.E.
Air Products
134,110 P.E.
Monsanto Co.
76,610 P.E.
Northeast S.T.P.
69,040 P.E.
American Cyanamid Co.
170,440 P.E.
-N-
PENSACOLA
TOTAL KJELDAHL NITROGEN WASTE SOURCES
ESCAMBIA BAY

Figure 6. Total Kjeldahl Nitrogen Waste Sources – Escambia Bay

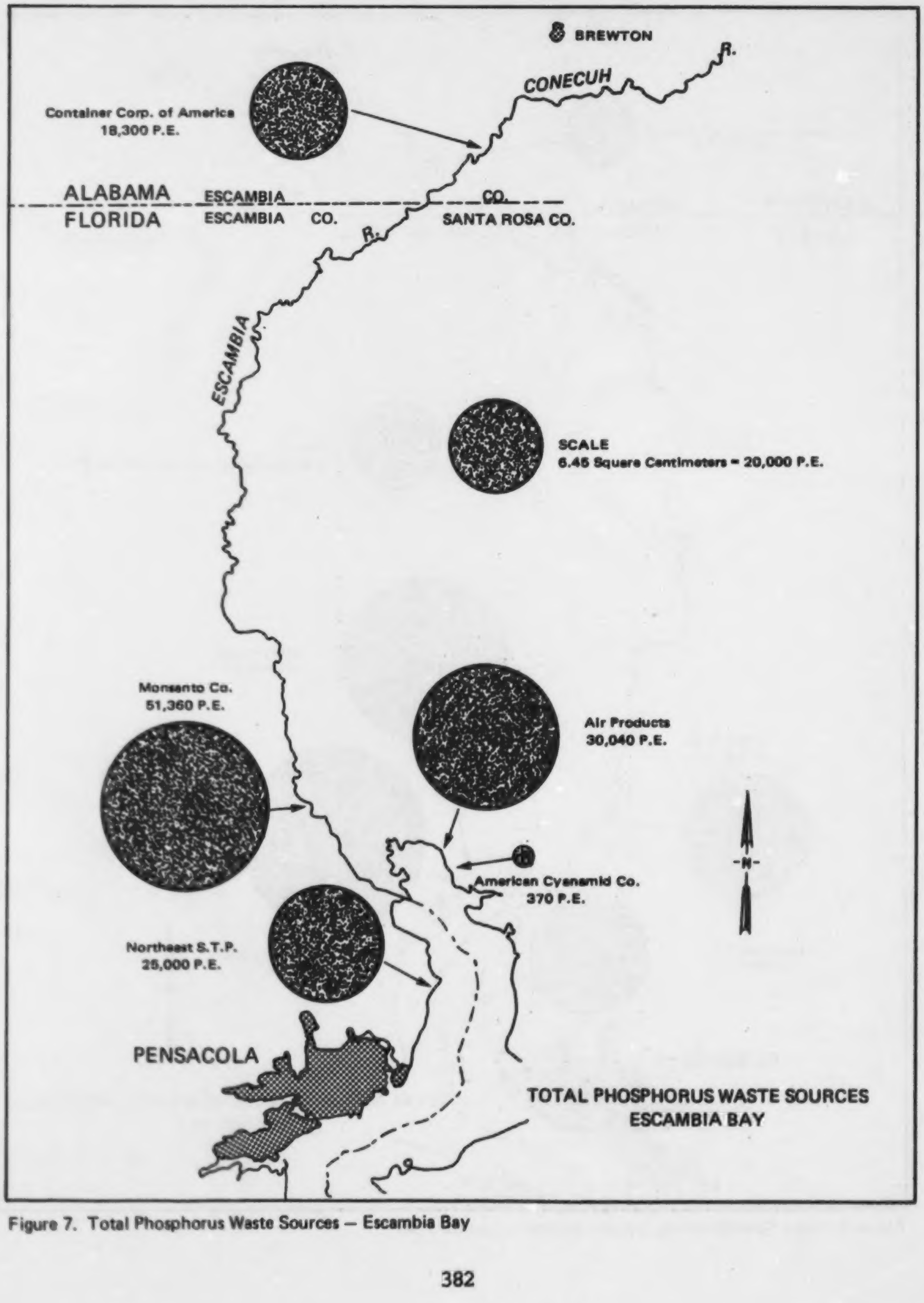
BREWTON
CONECUH
R.
Container Corp. of America
18,300 P.E.
ALABAMA
ESCAMBIA
CO.
FLORIDA
ESCAMBIA CO.
SANTA ROSA CO.
R.
ESCAMBIA
SCALE
6.45 Square Centimeters = 20,000 P.E.
Monsanto Co.
51,360 P.E.
Air Products
30,040 P.E.
American Cyanamid Co.
370 P.E.
N
Northeast S.T.P.
25,000 P.E.
PENSACOLA
TOTAL PHOSPHORUS WASTE SOURCES
ESCAMBIA BAY

Figure 7. Total Phosphorus Waste Sources – Escambia Bay

TABLE 2. ALLOWABLE WASTE DISCHARGE LOADING FOR MAJOR SOURCES OF POLLUTION

	CARBONACEOUS		NITROGENOUS			PHOSPHORUS	
SOURCE	5-Day BOD	TOC	TKN	NH_3-N	NO_2NO_3-N	Total	Ortho
American Cyanamid	425	500	249	32	42	–	–
Container Corp.	2230*	3120*	30	12	–	17	8
Escambia Chemical	17	12	197	137	143	35	35
Monsanto	605	282	102	66	80	46	29
Northeast STP	137	39	101	84	–	23	12
TOTAL	3414	3953	679	331	265	121	84

BOD = biochemical oxygen demand
TOC = total organic nitrogen
TKN = total Kjeldahl nitrogen
NH_3-N = ammonia nitrogen
NO_2, NO_3-N = nitrite-nitrate nitrogen

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* The contribution to Escambia Bay from this effluent will be 450 pounds per day of five-day BOD and 625 pounds per day of TOC.

now nearing operational status. Figure 8 shows the location of the unconsolidated facilities.

The AWT Plant in Pensacola, Florida is a physical-chemical treatment process. The plant receives industrial waste and is designed to treat both domestic and industrial waste. It should become operational within the next eight months.

CHEMICAL WATER QUALITY

Substantial water quality data was acquired in 1969. This 1969 data shows the state of water quality during the fish kills. TKN was at maximum concentrations in the northeast section of the Bay in the vicinity of the industrial outfalls. The surface value in this area was 1.9 mg/l, approximately three to four times more than in other areas of the Bay system. Nitrate Nitrogen was at a maximum in the northeast section where its concentration was ten times greater than in other areas of the Bay at a surface mean concentration of 1.16 mg/l.

Phosphate concentrations were highest in the northeast section with a mean concentration of 0.292 mg/l. The second highest concentration of 0.112 mg/l was observed just north of the Northeast STP.

Dissolved Oxygen (DO) in the Bay is dependent on several variables. The four major factors which may directly affect DO in Escambia Bay are: (1) stabilization of soluble or suspended oxygen demanding materials; (2) stabilization of bottom sediments containing high organic content; (3) photosynthesis and respiration by algae; and (4) reaeration.

Surface concentrations of DO were all measured above 5.0 mg/l in the upper Bay. In the lower Bay, samples were above 7 mg/l. There were wide swings in surface DO caused by algal photosynthesis. Wide swings in surface DO of up to 6.8 mg/l in a day were measured and indicates a highly enriched environment. Sediment demand on the DO of the water column causes a lower DO of roughly 4.0 mg/l. Conditions in the Bay unfavorable for fish are created due to the shallow nature of the Bay in the northeast section, the warm water temperatures, the algal respiration and the sediment demand.

The algal concentrations in Escambia Bay are shown in Figure 9. This may indicate that Mulatto Bay, a tributary to Escambia Bay and a focal point of several fish kills, acted as a trap for nutrients either because of the abandoned railroad tressel or dredging in the Bayou. The bottom of the Bayou is muck due to dredging where previously it was a grass bed. Reasons for high algal concentrations likely are the result of a highly complex physical and chemical synergism in and adjacent to the water column.

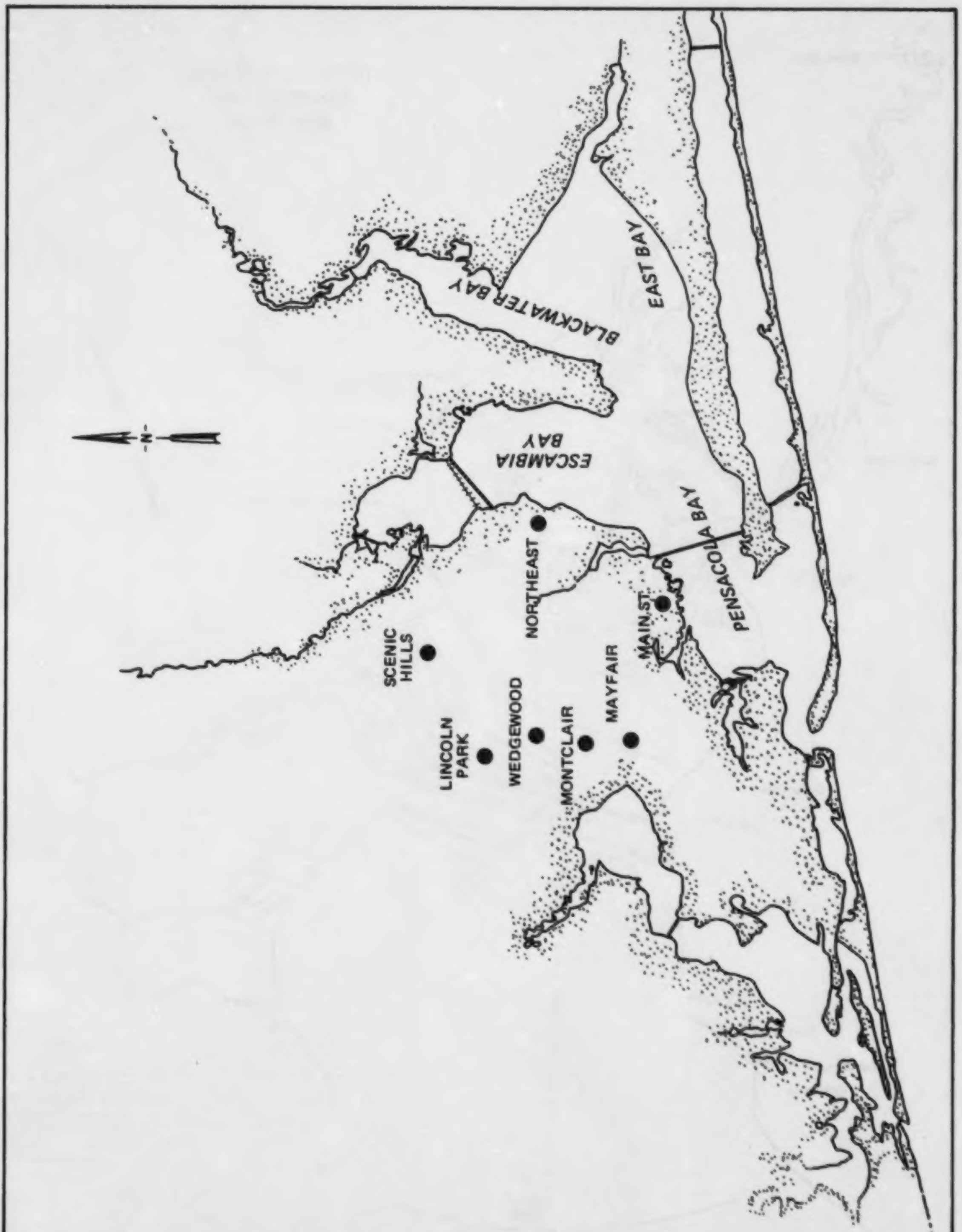

N
BLACKWATER BAY
EAST BAY
ESCAMBIA BAY
PENSACOLA BAY
NORTHEAST
MAIN ST.
SCENIC HILLS
LINCOLN PARK
WEDGEWOOD
MONTCLAIR
MAYFAIR

Figure 8. Municipal Waste Discharges – Escambia Bay System – 1970

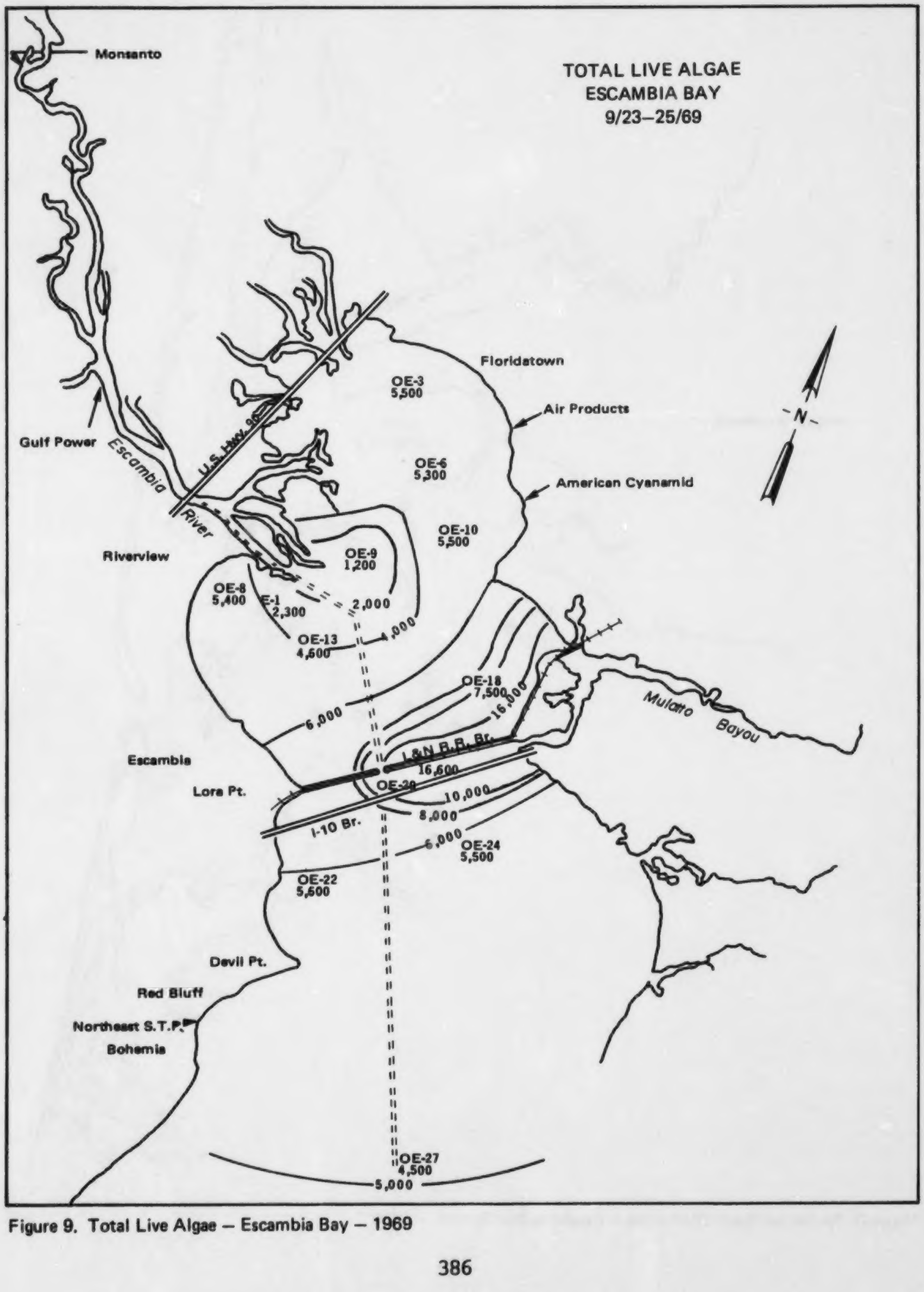

TOTAL LIVE ALGAE
ESCAMBIA BAY
9/23–25/69
Monsanto
Floridatown
Air Products
American Cyanamid
N
Gulf Power
Escambia River
U.S. Hwy. 90
Riverview
OE-3 5,500
OE-6 5,300
OE-10 5,500
OE-9 1,200
OE-8 5,400
E-1 2,300
2,000
4,000
OE-13 4,500
OE-18 7,500
16,000
Mulatto Bayou
6,000
Escambia
Lora Pt.
L&N R.R. Br.
16,600
OE-20
10,000
8,000
I-10 Br.
6,000
OE-24 5,500
OE-22 5,600
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
OE-27 4,500
5,000

Figure 9. Total Live Algae – Escambia Bay – 1969

The chemical water quality of the Escambia River and Escambia Bay over the last eight years gives few clues to lead one to believe that the water quality has been enhanced since 1969-1971. (See Figures 10, 11, 12.)

BIOLOGICAL WATER QUALITY

While not trying to give a totally accurate representation of the biological system, most of the studies in the period 1969-1974 pointed to problems. Fundamentally, it seems many of the problems are attributed to the disappearance of grass beds as a primary indicator of a major change in the overall ecosystem structure. Figure 13 shows the gradual progression of loss in the grass beds in Escambia Bay from 1949-1974. This long-term insidious alteration and resulting ecosystem disappearance is perhaps the major cause of loss of biological productivity. Likely causes include: (1) increased turbidity and siltation; (2) excessive shoaling and shifting of bottom substrate due to dredging activities; (3) simultaneously enhanced deposition of oxygen demanding organic sediments; and (4) the introduction of nutrients to the system resulting in a microphyte system instead of a macrophyte system. This results in grass bottom organisms being replaced by organisms adapted to the sludge bottom ecosystem. Oysters, shrimp, some fish and smaller organisms may be casualties of this change.

Data gathered by the Florida Department of Environmental Regulation (FDER) do not show dramatic trends when macroinvertebrates samples are measured versus time. (See Figures 14-20.) These figures may be interpreted in the following way:

Biotic Index

0 - 10	organic pollution
10 - 20	potential problems
> 20	absence of organic pollution

Diversity Index

< 1	poor biological community
1 - 3	fair to good biological community
> 3	stable and diverse biological community

This lack of a dramatic trend may be the result of having insufficient data or it may show the lack of comparative data during highly polluted stages. Two other alternatives are less reassuring. Since the first two attribute the problem to data acquisition, these latter two point to other problems. The third alternative suggests a requirement for a very long time to lapse before the system again becomes fully healthy and the fourth would indicate that once destroyed, a marine estuary

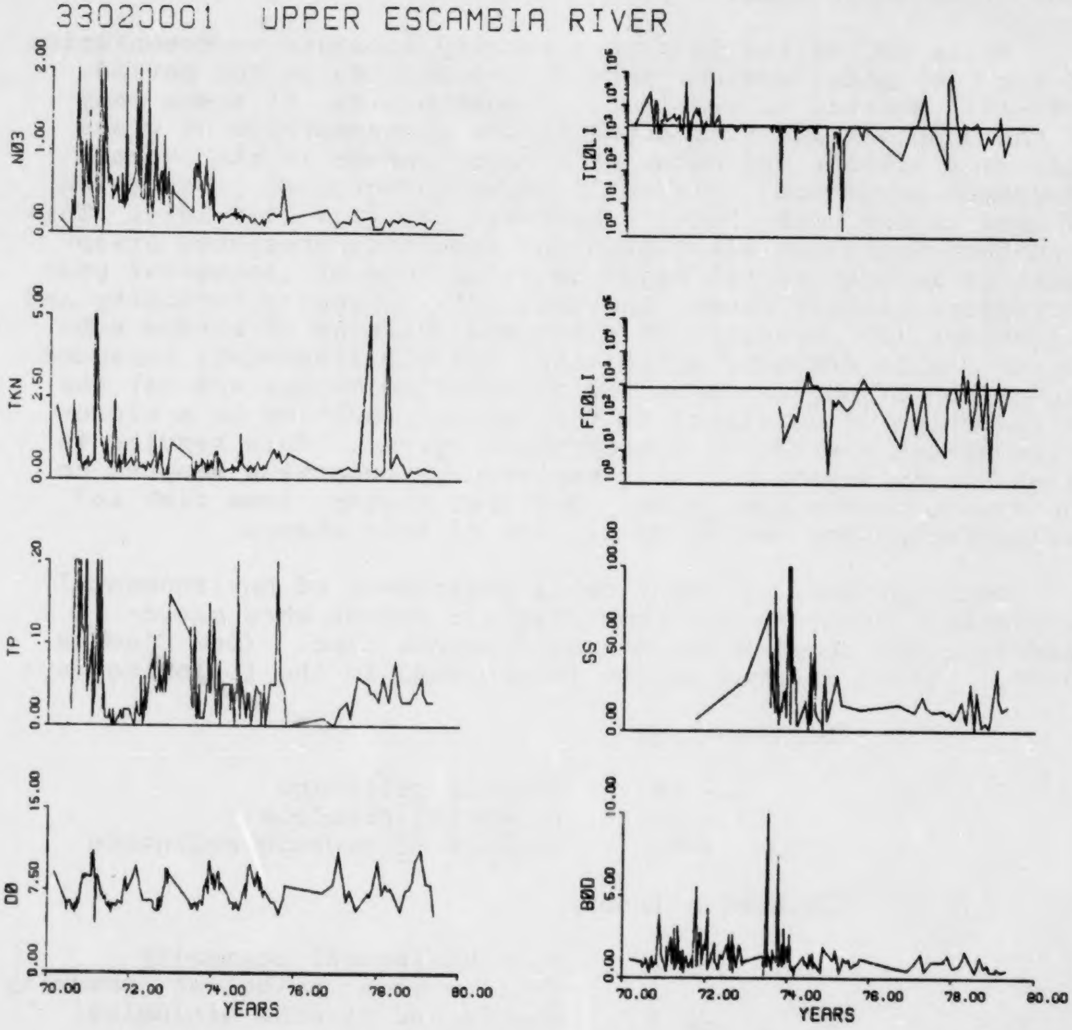


Figure 10. Water Quality – 8 Parameters – 1970-1979

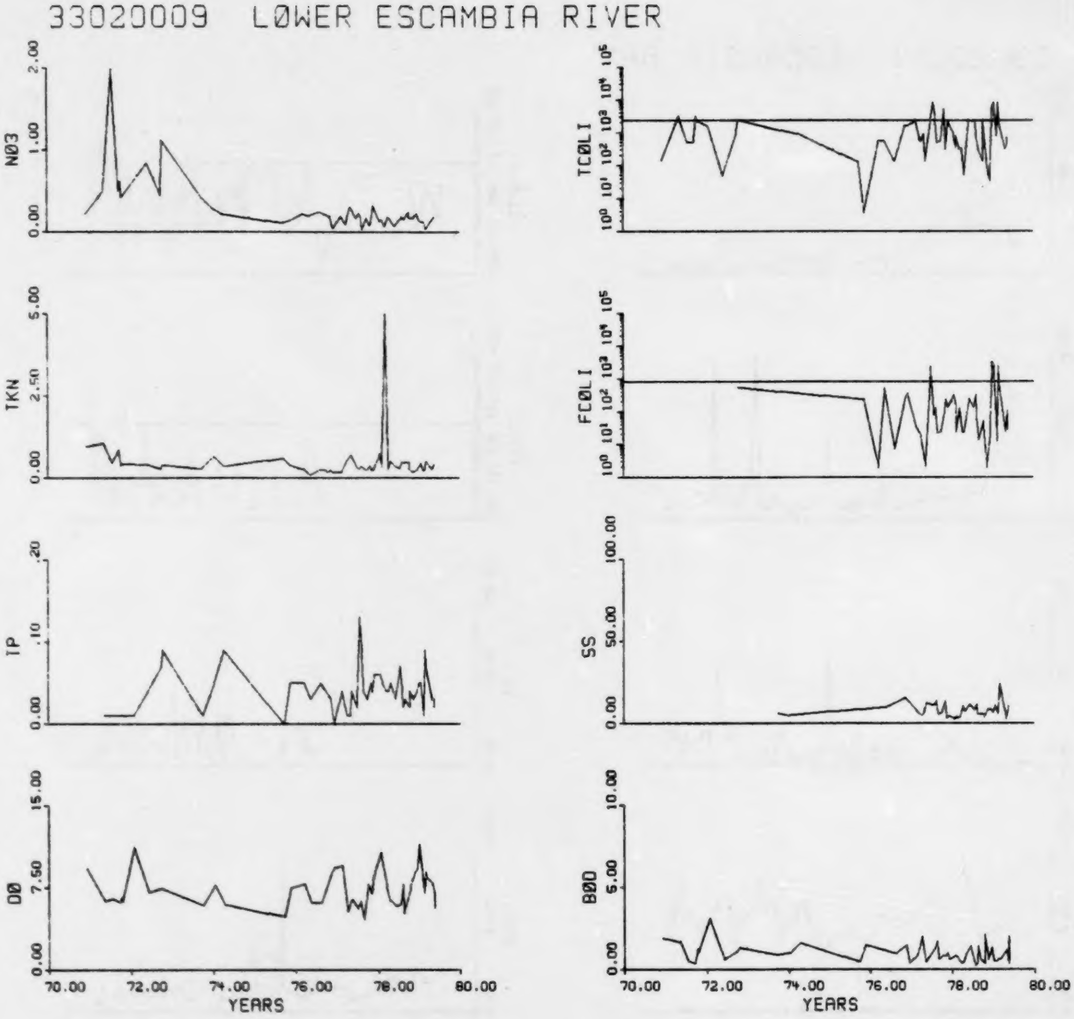


Figure 11. Water Quality — 8 Parameters — 1970-1979

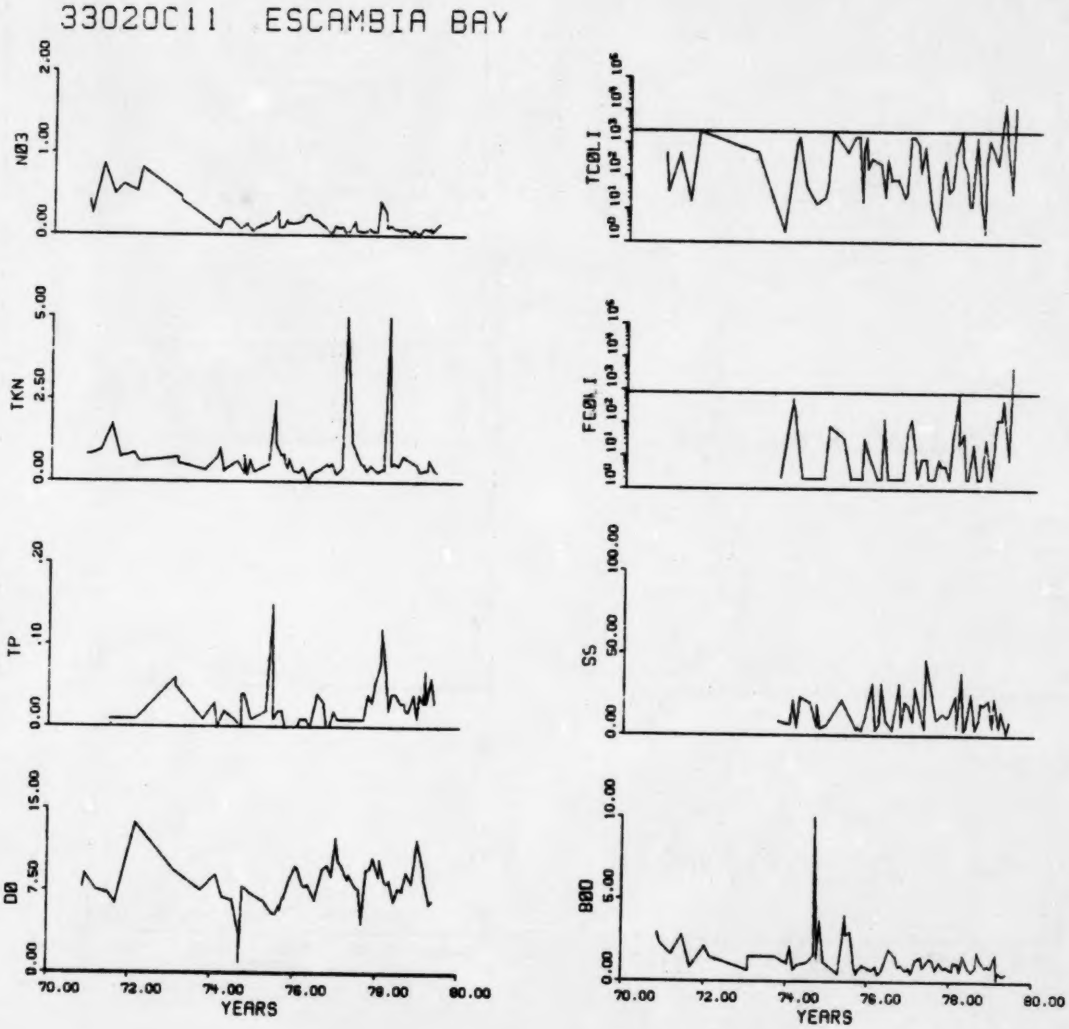


Figure 12. Water Quality — 8 Parameters — 1970-1979

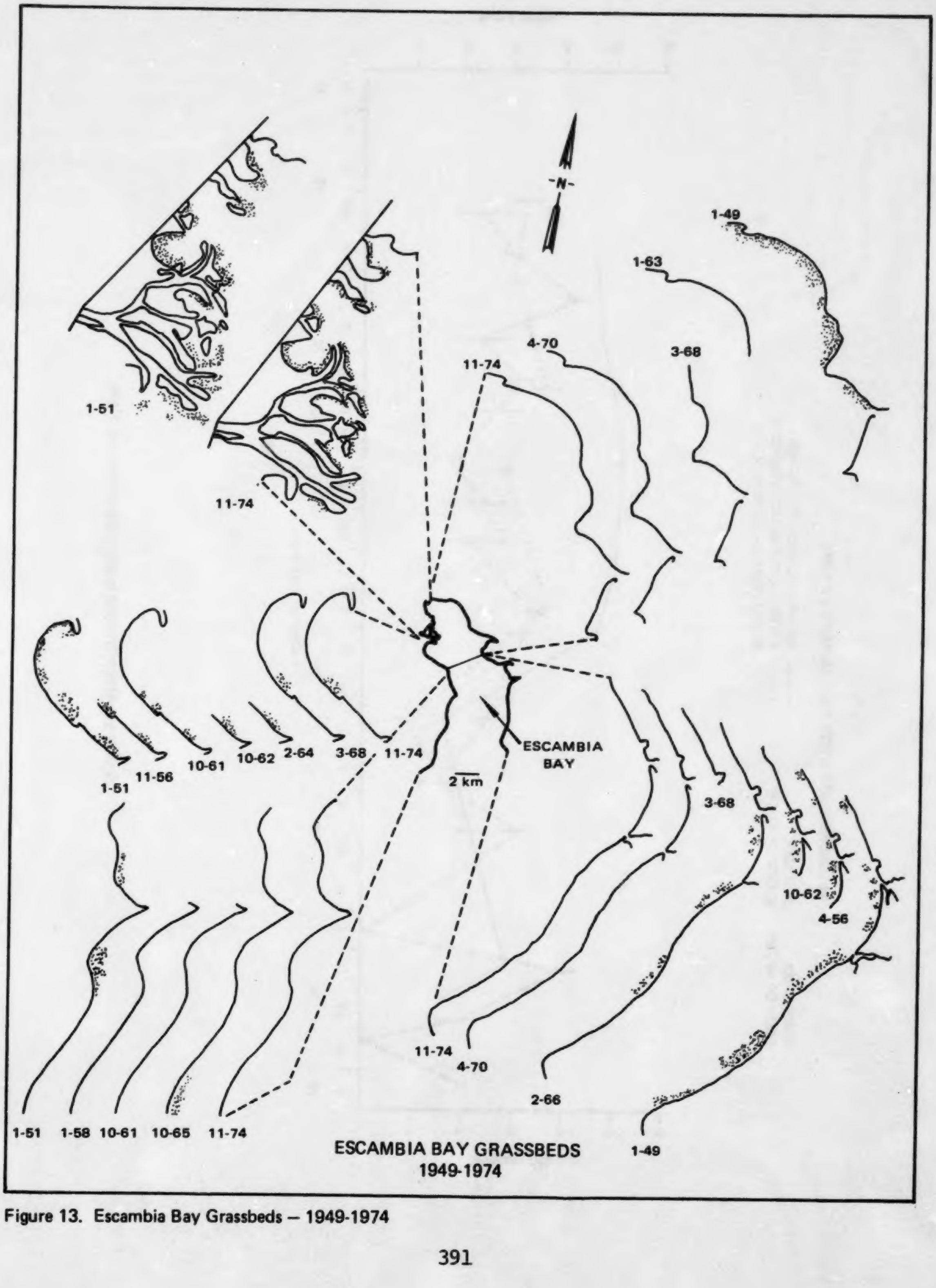

-N-
1-51
11-74
1-49
1-63
4-70
11-74
3-68
1-51
11-56
10-61
10-62
2-64
3-68
11-74
ESCAMBIA
BAY
2 km
3-68
10-62
4-56
11-74
4-70
2-66
1-49
1-51
1-58
10-61
10-65
11-74
ESCAMBIA BAY GRASSBEDS
1949-1974

Figure 13. Escambia Bay Grassbeds – 1949-1974

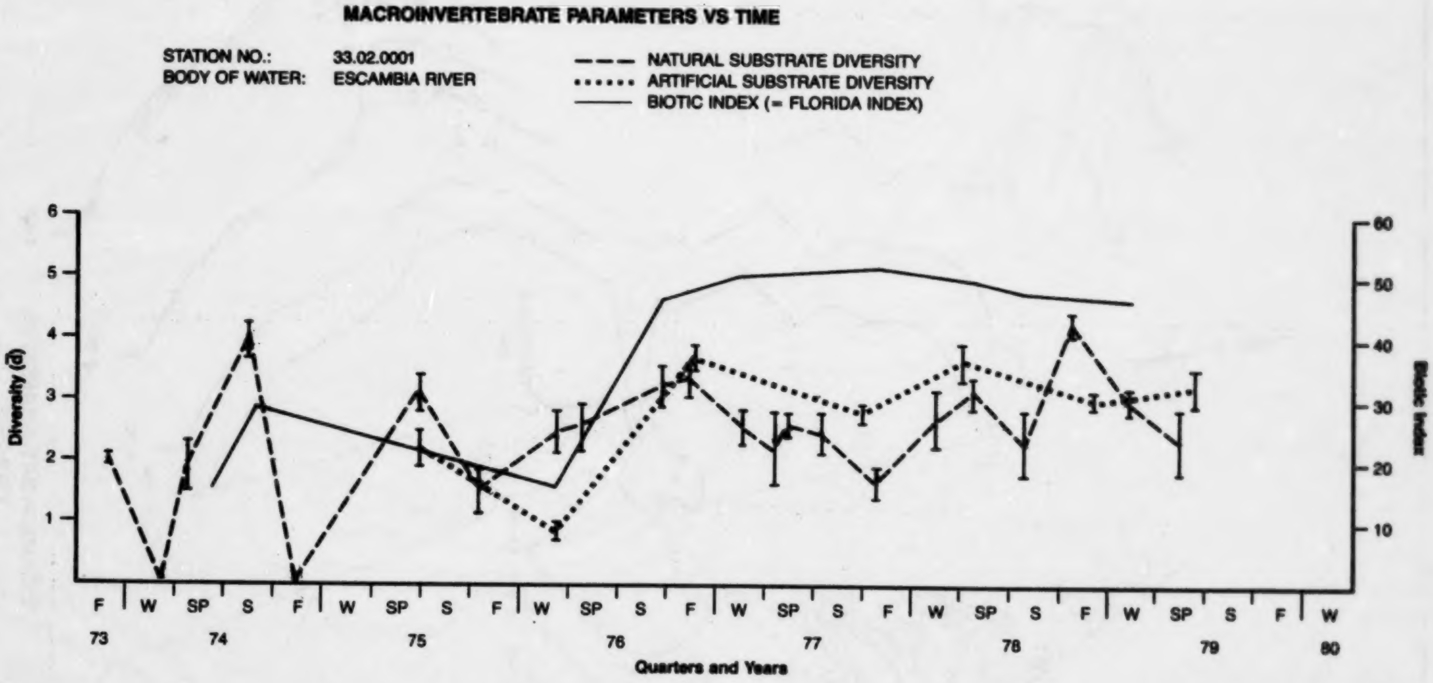


Figure 14. Macroinvertebrate parameters vs time.

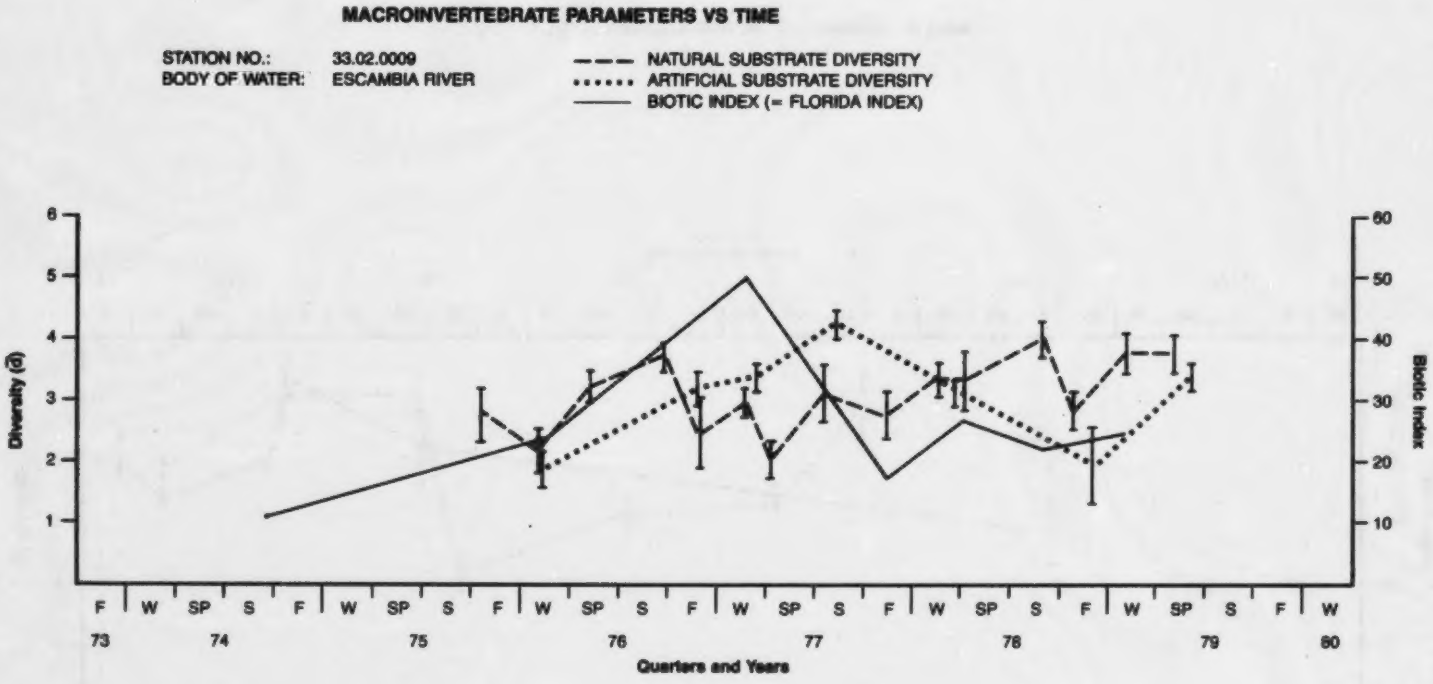


Figure 15. Macroinvertebrate parameters vs time.

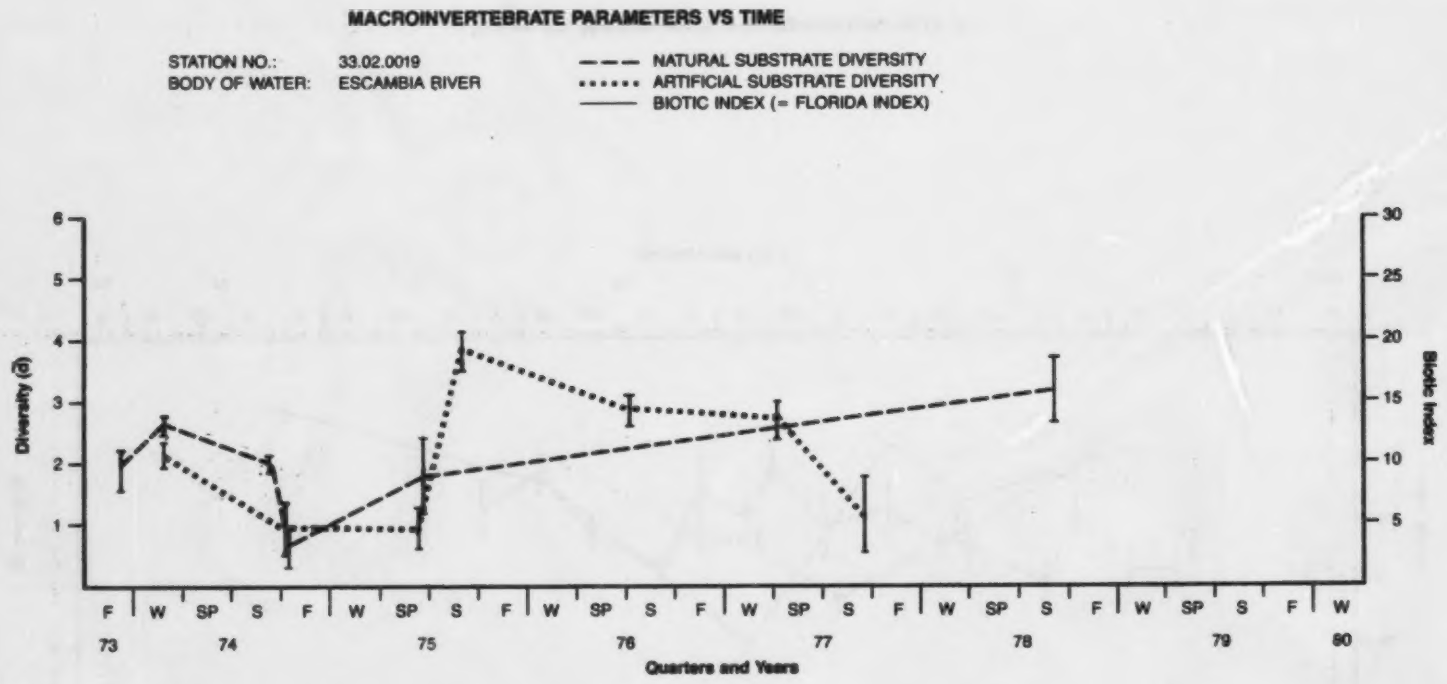


Figure 16. Macroinvertebrate parameters vs time.

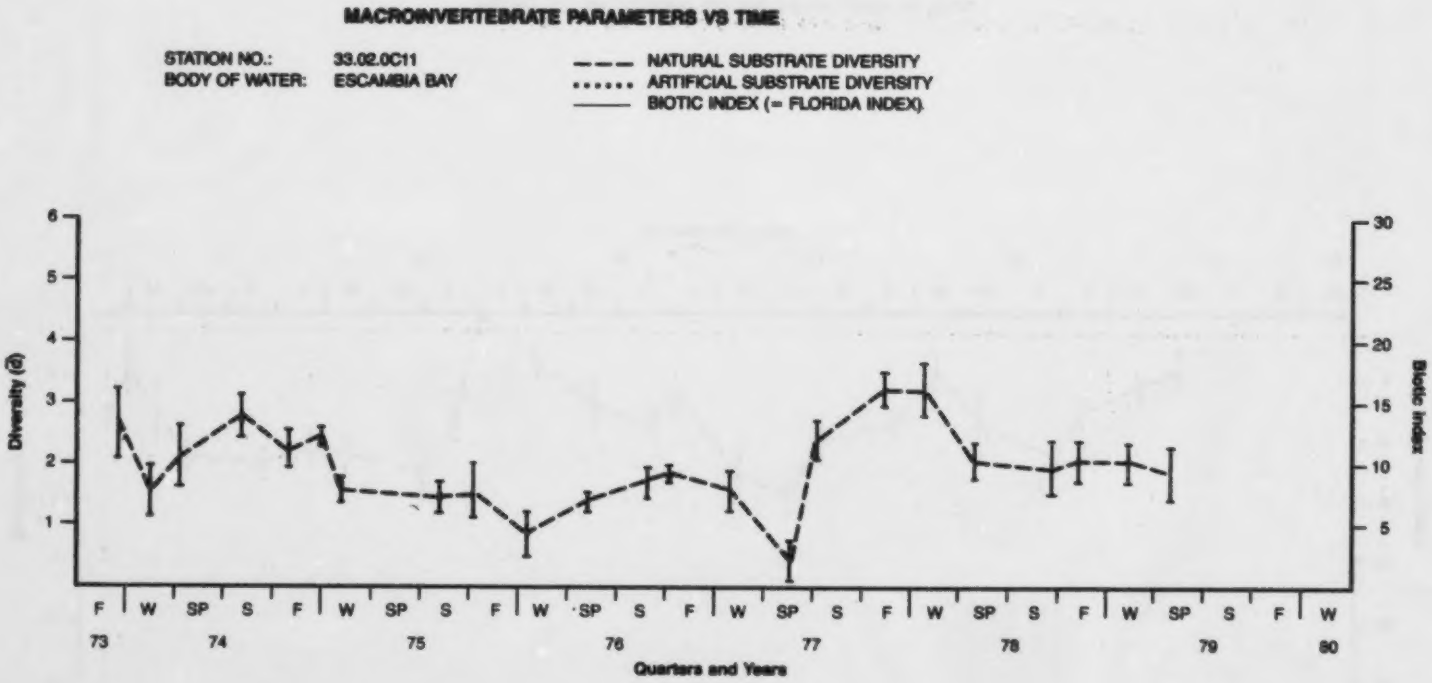


Figure 17. Macroinvertebrate parameters vs time.

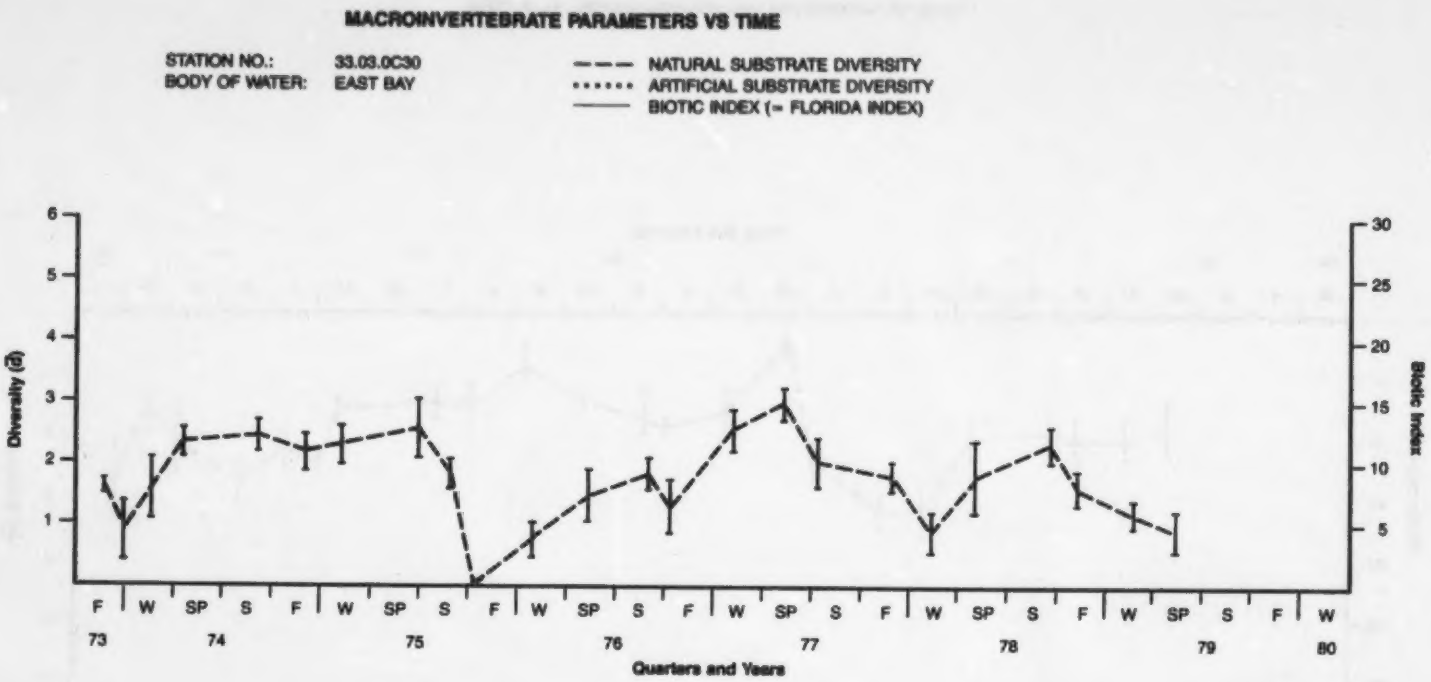


Figure 18. Macroinvertebrate parameters vs time.

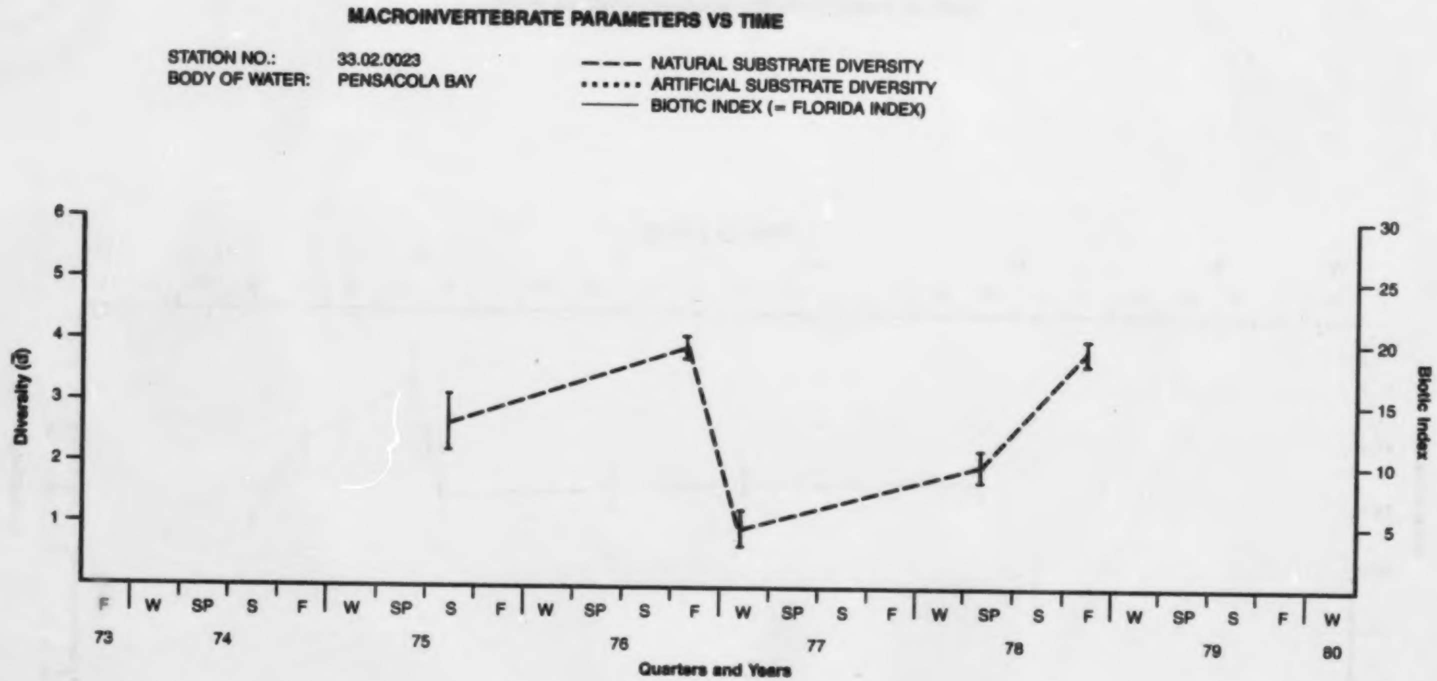


Figure 19. Macroinvertebrate parameters vs time.

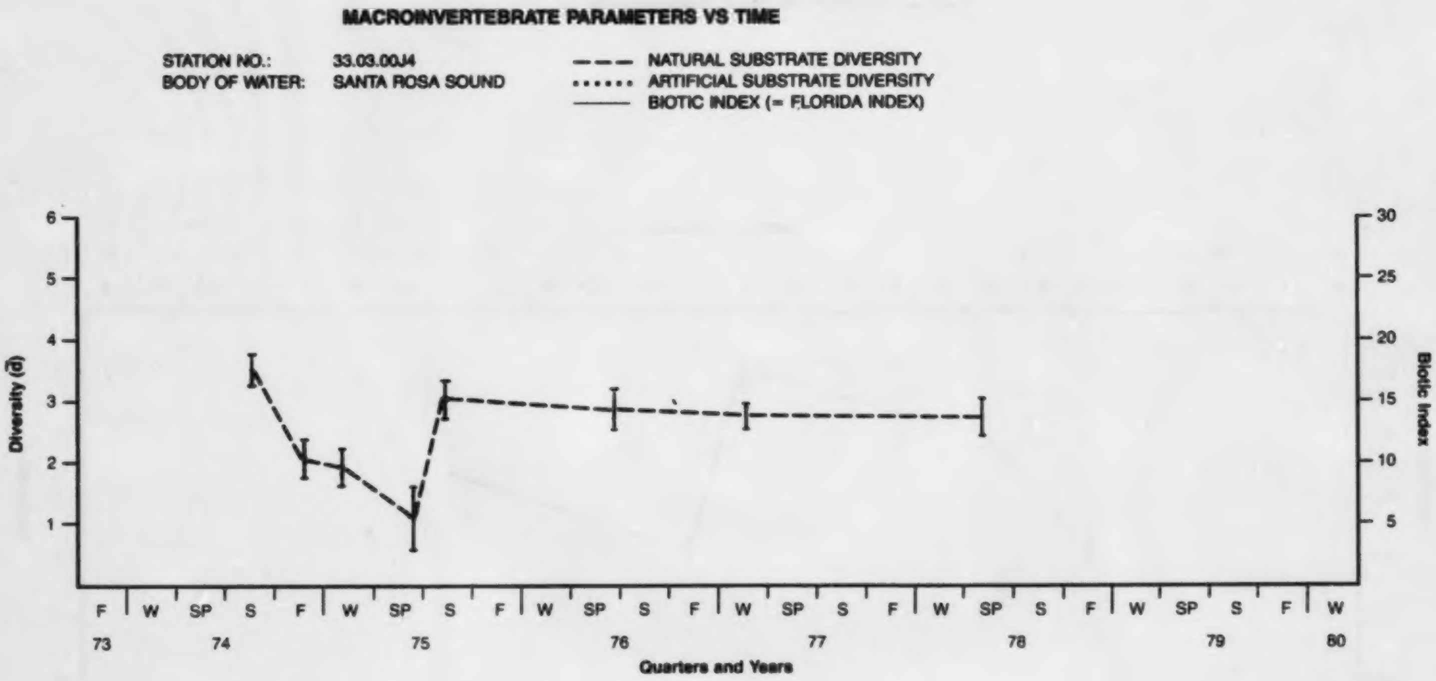


Figure 20. Macroinvertebrate parameters vs time.

would require several lifetimes for recovery.

Fortunately, the first two alternatives seem to be the more likely ones. After the big fish kills in 1969-1971 and the resulting clean up by the industries, fish kills have been minimal and the fish population has increased. Since the early part of the 1970's, some grass beds have begun to re-emerge. Fish, shrimp and oysters have begun to repopulate the cleaner fringes of the Bay showing that the clean up was effective.

SEDIMENT

Alluded to earlier, sediment is a real problem in the Bay system. Unconsolidated sediments range in depth from .6 meters or less to 2 meters. Approximately one-third of the Bay is covered with greater than 2 meters of sediment. (See Figure 21.) Comparison of Escambia Bay sediments with those of an adjacent bay, show the sediment is rich in carbonaceous, phosphorus and nitrogenous material with relative proportions as follows:

	Escambia Bay	East Bay
C	45.4	54
N	5	.75
P	1	1

Thus, as the ecological system sees these nutrients, nitrogen is 6.75 times larger in Escambia Bay than East Bay in the same system.

Some studies performed by the Federal Water Quality Administration in 1970 reveal concentration patterns of these nutrients in the sediments. Figures 22, 23, 24 show the concentration patterns for each nutrient. This sediment load could continue to have detrimental effects for many years after the point sources are cleaned up.

Griffin (1972) stated that the bay system is practically isolated from the Gulf of Mexico even though there is a physical connection and the Bay is tidally influenced. This determination was based on findings that showed the clay, mortimorillonite, found in Pensacola Bay in abundance was not found in the sediment shelf of the Gulf. Other bay systems did show a commonality of depositions in the Gulf and in the Bay. He did find that the clay migrated into Escambia Bay. This shows that man's activity within Pensacola Bay influences Escambia Bay more than one would have guessed. It thus appears that essentially all of the fine grained inorganic detritus going into the Escambia-Pensacola Bay system will be deposited within that system. Thus, the bay system may not flush as well as might be suggested by the tidal influence or freshwater introduction. Dredging and sewage discharges in Pensacola Bay thus have an impact on Escambia Bay.

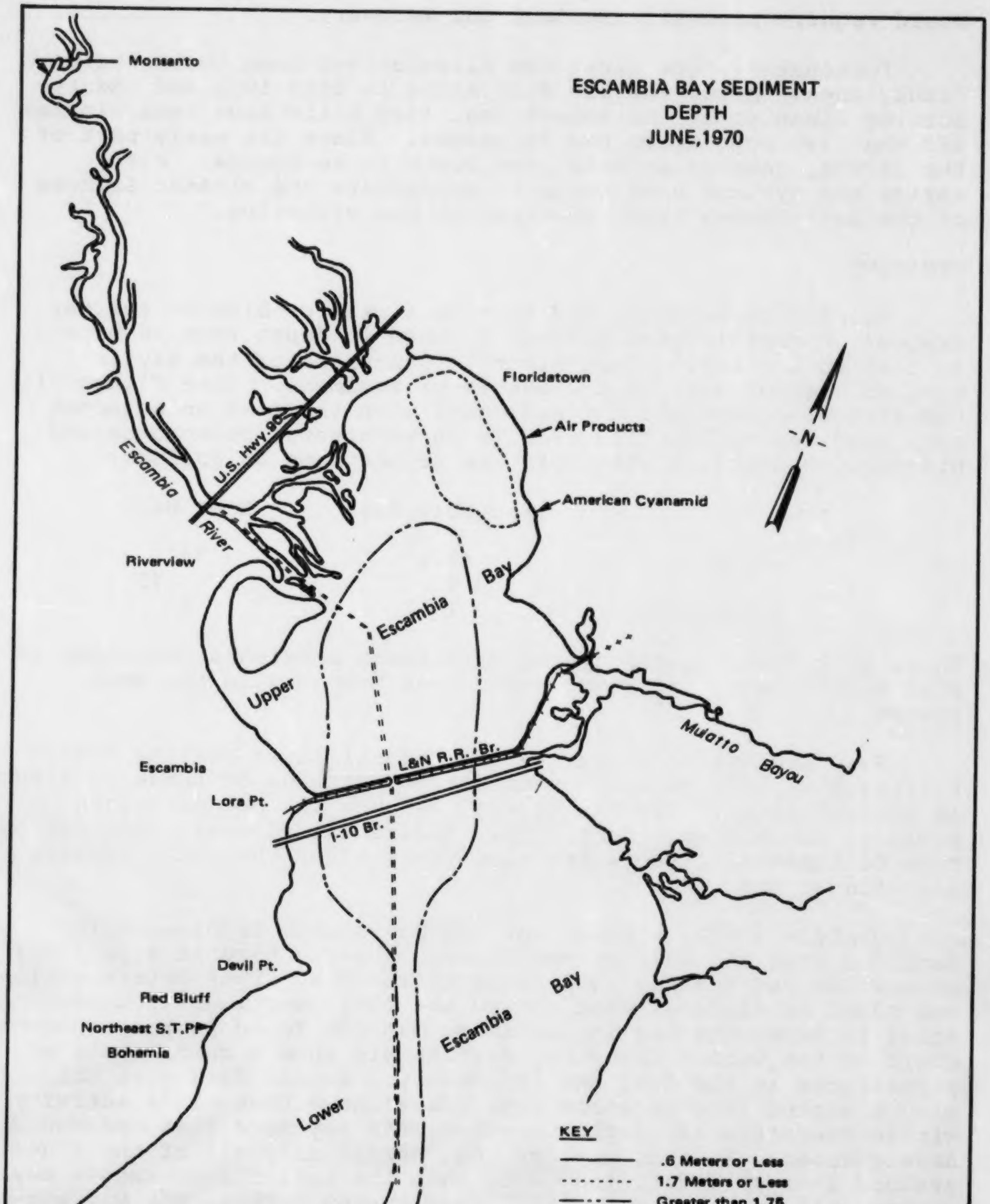
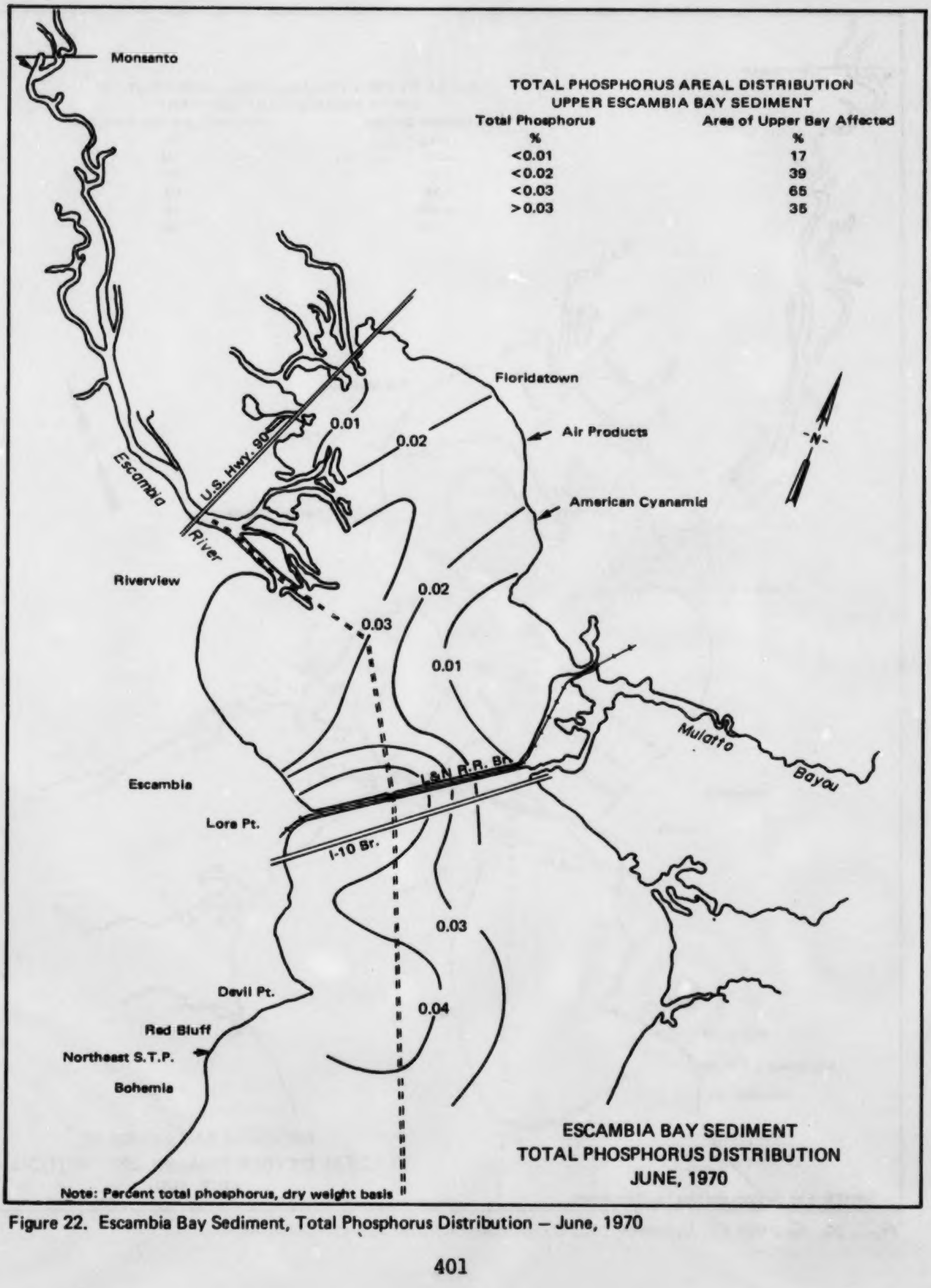

ESCAMBIA BAY SEDIMENT
DEPTH
JUNE,1970
Monsanto
Floridatown
Air Products
American Cyanamid
N
U.S. Hwy. 90
Escambia
River
Riverview
Escambia
Bay
Upper
Mulatto
Bayou
Escambia
Lora Pt.
L&N R.R. Br.
I-10 Br.
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
Bay
Escambia
Lower
KEY
.6 Meters or Less
1.7 Meters or Less
Greater than 1.75
Note: Meters of Sediment

Figure 21. Escambia Bay Sediment Depth — June, 1970


Monsanto
Floridatown
Air Products
American Cyanamid
U.S. Hwy. 90
Escambia River
Riverview
0.01
0.02
0.02
0.03
0.01
Escambia
Lora Pt.
L&N R.R. Br.
I-10 Br.
Mulatto Bayou
0.03
Devil Pt.
0.04
Red Bluff
Northeast S.T.P.
Bohemia
N

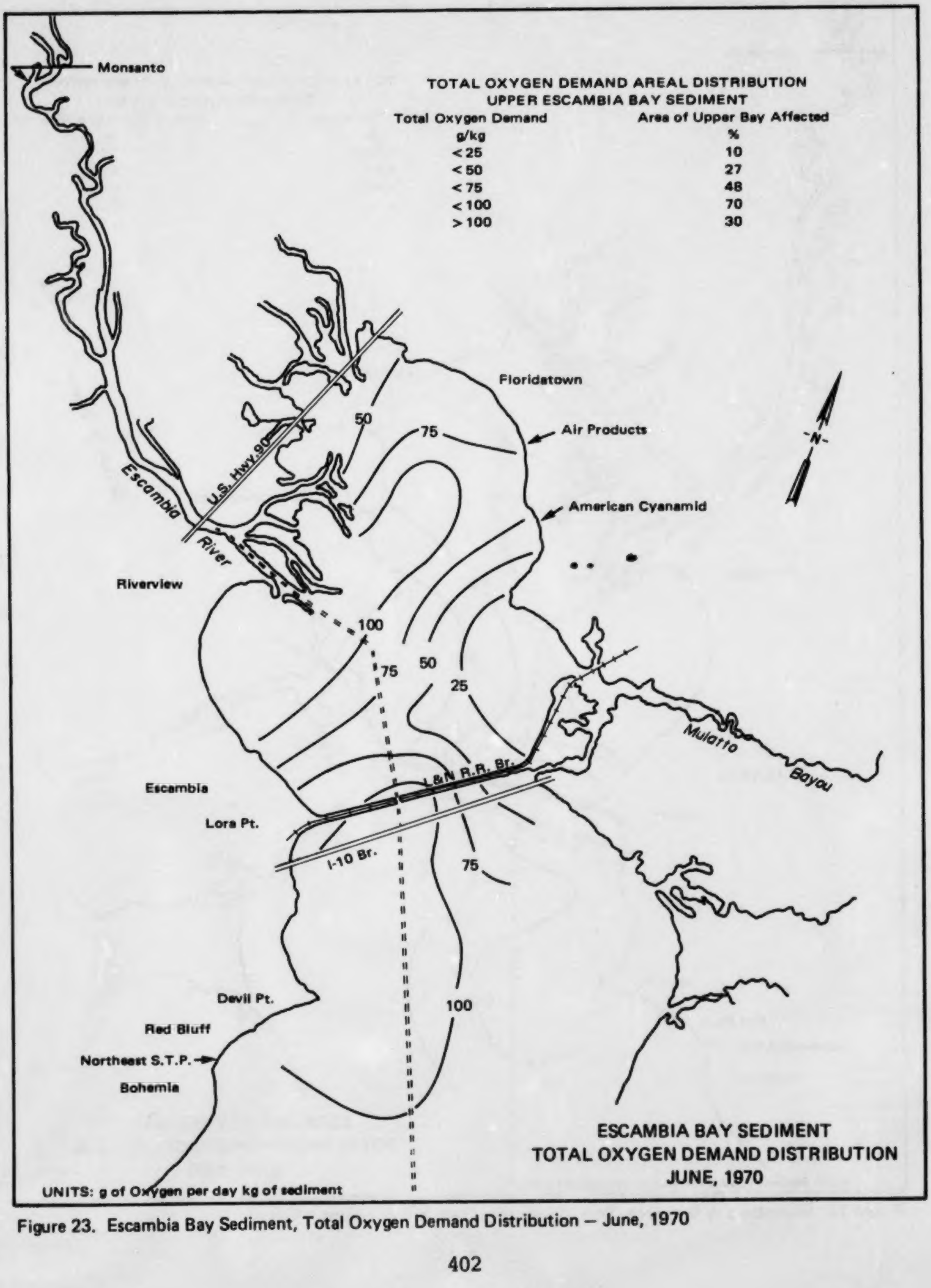
TOTAL PHOSPHORUS AREAL DISTRIBUTION
UPPER ESCAMBIA BAY SEDIMENT

Total Phosphorus %	Area of Upper Bay Affected %
<0.01	17
<0.02	39
<0.03	65
>0.03	35

ESCAMBIA BAY SEDIMENT
TOTAL PHOSPHORUS DISTRIBUTION
JUNE, 1970

Note: Percent total phosphorus, dry weight basis

Figure 22. Escambia Bay Sediment, Total Phosphorus Distribution – June, 1970


Monsanto
Floridatown
Air Products
American Cyanamid
-N-
U.S. Hwy.90
Escambia River
Riverview
50
75
100
75
50
25
Mulatto Bayou
L&N R.R. Br.
Escambia
Lora Pt.
I-10 Br.
75
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
100

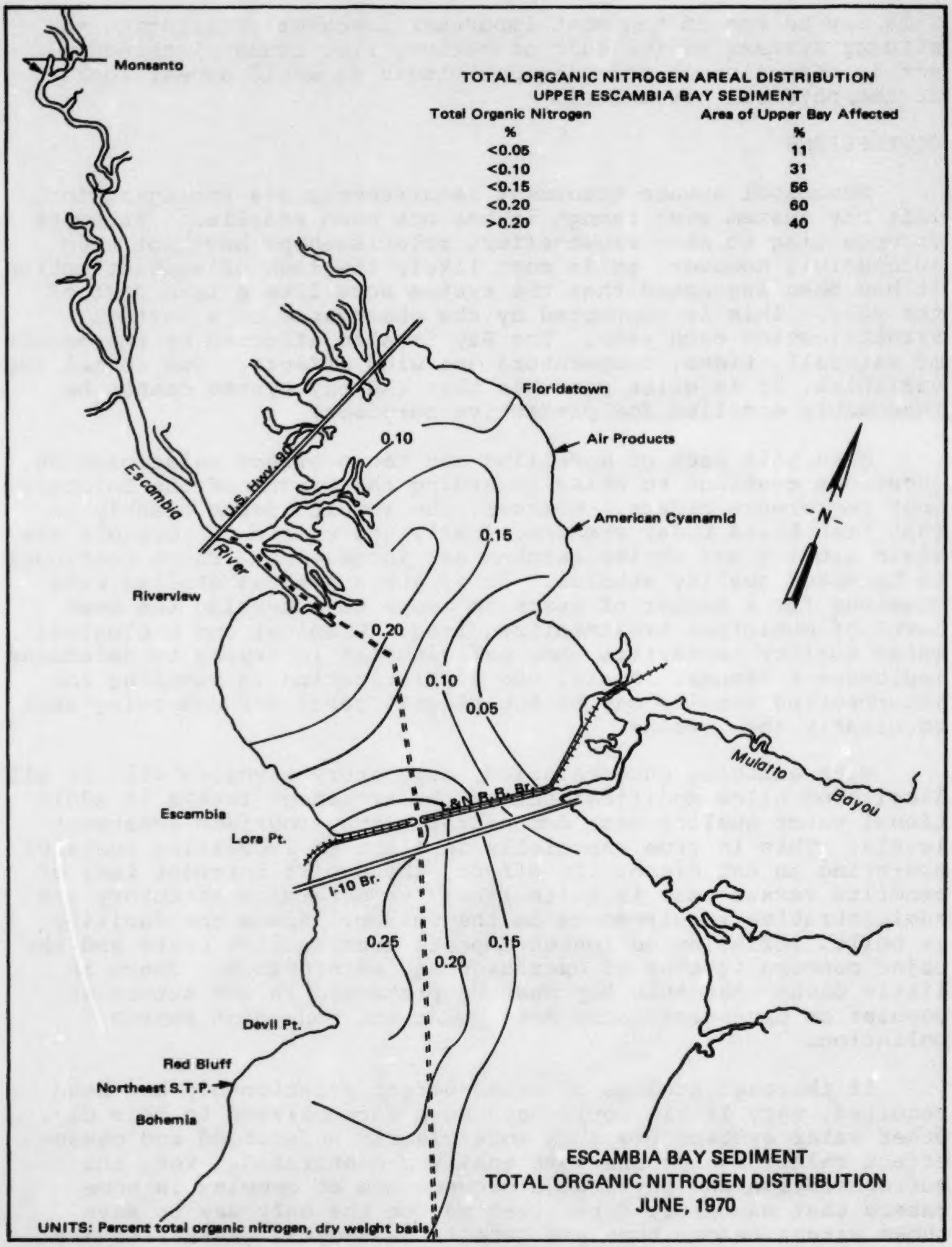
TOTAL OXYGEN DEMAND AREAL DISTRIBUTION
UPPER ESCAMBIA BAY SEDIMENT

Total Oxygen Demand g/kg	Area of Upper Bay Affected %
< 25	10
< 50	27
< 75	48
< 100	70
> 100	30

ESCAMBIA BAY SEDIMENT
TOTAL OXYGEN DEMAND DISTRIBUTION
JUNE, 1970

UNITS: g of Oxygen per day kg of sediment

Figure 23. Escambia Bay Sediment, Total Oxygen Demand Distribution – June, 1970


Monsanto
Floridatown
Air Products
American Cyanamid
U.S. Hwy. 90
Escambia
River
Riverview
0.10
0.15
0.20
0.10
0.05
Mulatto
Bayou
L&N R.R. Br.
Escambia
Lora Pt.
I-10 Br.
0.25
0.20
0.15
Devil Pt.
Red Bluff
Northeast S.T.P.
Bohemia
N
ESCAMBIA BAY SEDIMENT
TOTAL ORGANIC NITROGEN DISTRIBUTION
JUNE, 1970
UNITS: Percent total organic nitrogen, dry weight basis

TOTAL ORGANIC NITROGEN AREAL DISTRIBUTION
UPPER ESCAMBIA BAY SEDIMENT

Total Organic Nitrogen %	Area of Upper Bay Affected %
<0.05	11
<0.10	31
<0.15	56
<0.20	60
>0.20	40

Figure 24. Escambia Bay Sediment, Total Organic Nitrogen Distribution – June, 1970

This may be one of the most important discoveries relating to estuary systems in the Gulf of Mexico, i.e. tidal flushing is not as effective in removing pollutants as would appear looking at the physical parameters.

CONCLUSIONS

Municipal sewage treatment requirements are stringent for this bay system even though it has not been modelled. Attempts at modelling to show cause-effect relationships have not been successful; however, it is most likely the lack of sophistication. It has been suggested that the system acts like a lake part of the year. This is supported by the observance of a partial stratification each year. The Bay is also affected by the amount of rainfall, tides, temperature and wind effects. Due to all the variables, it is quite possible that the bay system cannot be reasonably modelled for predictive purposes.

With this lack of modelling and cause-effect relationships, questions continue to arise regarding the wisdom of the Enforcement Conference orders. However, the result speaks clearly in that fish kills today are practically non-existent. Oysters are again growing and shrimp catches are increasing. There continues to be water quality studies. It is certain these studies will continue for a number of years in order to determine the best level of municipal treatment required. Chemical and biological water quality parameters show deficiencies in trying to determine improvement trends. Again, our sophistication in sampling and interpreting results may be the biggest cause for not being able to clearly show trends.

With existing uncertainties, regulatory agencies will in all likelihood allow modifications to the treatment levels if additional water quality data demonstrates inappropriate treatment levels. This is true especially in light of increasing costs of operating an AWT Plant. In effect, the public interest test of benefits versus cost is quite likely to determine statutory and administrative requirements in the future. Since the facility is built, inflation no longer impacts construction costs and the major concern is that of operation and maintenance. There is little doubt that this Bay must be protected in the future as population pressures cause more point and non-point source pollution.

If thorough studies of cause-effect relationship had been required, very little would have been accomplished to this day. Other water systems are much more clearly understood and cause-effect relationships are more easily demonstrated. Yet, the eutrophication and physical processes are so complex in some waters that mandatory directives may be the only way to save those waters before they are totally destroyed. Estuaries are extremely sensitive and fragile systems.

Recognizing the difficulties of predicting and showing cause-effect relationships, this bay system clearly was in a horrible state in 1970. Today the situation is dramatically different. No shrimp were found in quantity in the system until the spring of 1976. In the summer of 1977, profitable catches of shrimp were reported in Escambia Bay as far north as the interstate bridge. Speckled trout were also being caught in good numbers. Oyster beds and grass beds are being re-established. Larvel oysters and small oysters are being found in large quantities in the Bay since the total kill in 1971 and partial kill in 1974-1975. Grasses, mostly ulgrass (Vallisneria Americana), are spreading along the river mouth and northwest shoreline of Escambia Bay and other parts of the bay system less affected by earlier pollution.

In conclusion, it must be stated that had the regulatory agencies done nothing, the improved conditions might exist today. This seems highly unlikely; however, there is no way to prove otherwise. Since no pre-monitoring can be compared with post-monitoring, cause-effect relationships are hard to establish. It is clear that problems existed in the Bay before regulatory action and that those problems have been mitigated after the regulatory action. This suggests a direct relationship between the problem and the subsequent action. As late as 1975, the U.S. EPA still recommended that the sewage discharges to the Escambia-Pensacola Bay system be limited and AWT was still recommended for the Pensacola Main Street Plant. The FDER has continued to support this recommendation due to the poor circulation and continuing problems in some of the bayous tributary to Escambia Bay and Pensacola Bay.

SECTION 3

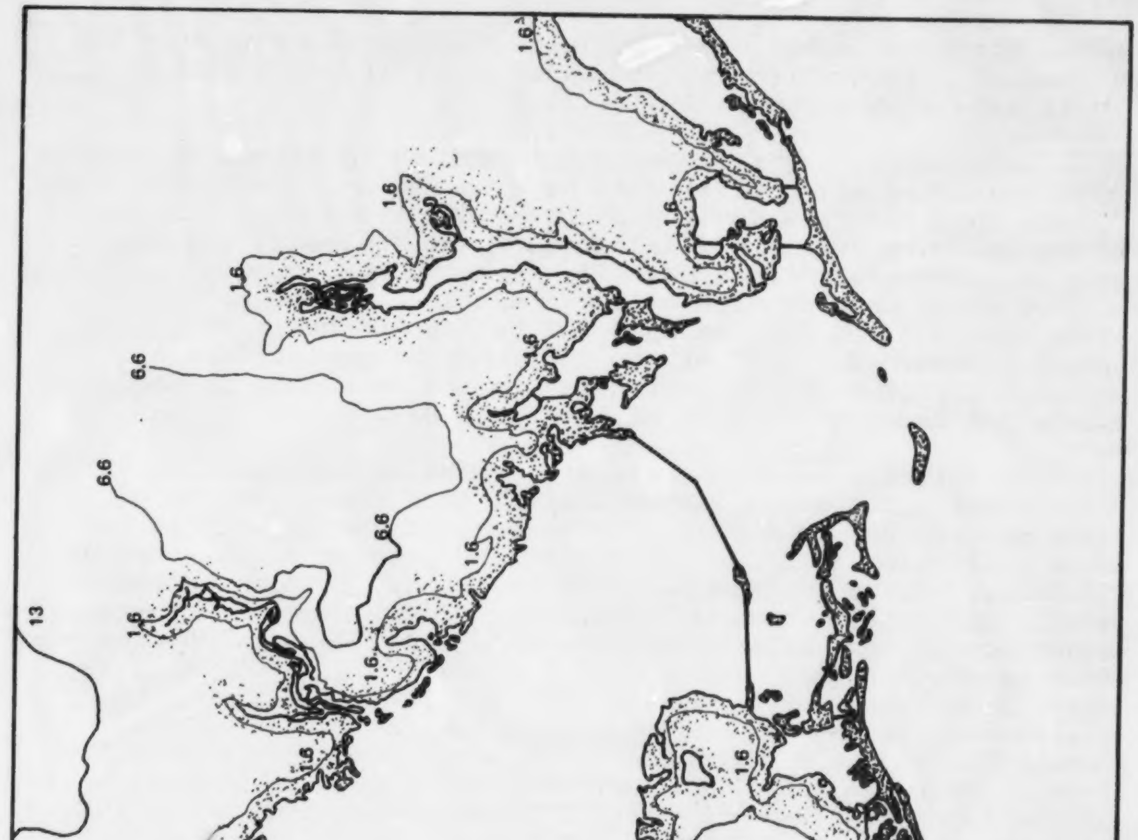
TAMPA BAY

PHYSICAL CHARACTERISTICS AND HISTORY

Tampa Bay lies on the west-central coast of Florida and is composed of several sub-bays, all of which jointly constitute Tampa Bay. Area rainfall averages about 122 centimeters per year, 50 percent of which occur in the summer months of June to September. However, as in the Escambia Bay area, hurricanes can have drastic impacts on the amount of rain the area experiences, sometimes to levels of 25-50 centimeters in a few days time. The topography is extremely flat with elevations exceeding no more than 6-8 meters at the highest points. (See Figure 25.) Depths in the bay system range up to 10 meters except in the Tampa ship channel which is today being deepened to 14 meters. The tide's diurnal range is 1 meter.

The Tampa Bay system has been recognized as having a pollution problem since the early 1960's. In 1965, state and local governments studied the problem of odors caused by decaying algae in the western part of Hillsborough Bay. Housekeeping was generally recommended, i.e. removal of the algae. This in retrospect was a stop gap measure and little was done by state, local or federal officials until the late 1960's when a federal study (June 1967) known as the "Hagan Report" was conducted (Hagan 1969). During the late 1960's, local government called on state and federal authorities to help solve the pollution problems in the Bay. This study identified the problems occurring in the bay system and proposed solutions. Now roughly ten years later, a number of these recommendations have been completed, one of which was the construction of an AWT Plant for the City of Tampa. The results have not been fully evaluated, but there are some improvements. Dramatic results were envisioned when regulatory orders for treatment were first issued. These have not occurred. Perhaps the bay system has not had ample time to recover and only time will tell if the corrections really worked. Bay sediments, dredge and fill activities and stormwater runoff may also have more impact on the Bay than early regulatory activities envisioned.

Within Tampa Bay, the currents have been studied extensively by Dr. Bernard Ross, Professor at the University of South Florida. He has modelled the entire Bay and it is fairly well known, based on the model, what happens to a distinct particle released to the


1.6
1.6
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13
1.6
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13
1.6
6.6
6.6
1.6
5
5
1.6
1.6
5
5
1.6
1.6
3.3
3.3
8.3
1.6
5
8.3
8.3
8.3
5
1.6
1.6
8.3
8.3
1.6
1.6
1.6

Figure 25. Topography of the Tampa Bay Land Area

Bay. Dissolved substances have also been modelled to show the impact of dilution from tidal mixing and rainfall. (These currents behave as shown in Figure 26.)

This model is very important in trying to assess the impact that wasteload allocations will have on the bay system for regulatory purposes. In developing the model, 1254 neutrally buoyant particles were spaced one-half mile apart, released and studied for location every ten minutes for thirty days. The accompanying figure shows the location of these particles after thirty days. (See Figure 27.) Many escaped the bay system but significant numbers remained. Most of the particles accumulated at a spot on the Gulf side of the peninsula, in the main ship channel, in upper Old Tampa Bay and in Hillsborough Bay.

Dissolved substances were studied using non-reactive tracer substances. The study lasted until a seed concentration of 10 mg/l was reduced to 1 mg/l in each area of the Bay. This was done by looking at these concentrations through monthly cycles of tides. Based on this phase of the study, it took approximately 180 days to have a 70 percent reduction in the non-reactive substance in the water column of Tampa Bay. Ross calculates that 25.8 percent of the tidal quantity of water is permanently removed from Tampa Bay by mixing with the Gulf of Mexico. He claims that as the tide leaves Tampa Bay, it carries dissolved material. When the water exits Tampa Bay, it mixes with Gulf water. When the new tide comes into Tampa Bay, it returns much of the same dissolved substance. Indeed, the model shows that 74.2 percent of the dissolved substance returns from the Gulf.

As is true in most bay systems in Florida, freshwater has a significant effect on the predominantly saltwater ecosystems. Sources of freshwater in Hillsborough Bay are predominantly the three rivers: Hillsborough, Alafia and Palm Rivers. They act on the bay system as if they were point source discharges. In the case of the Alafia River, it is both a source of freshwater and a major source of phosphate pollution. During its flow, it acquires phosphate from natural sources and phosphate mine discharges. The drainage basin for these rivers is the source of most of the United States' phosphate fertilizers. Thus, the addition of phosphates to Tampa Bay is not unexpected.

In the early days of Tampa and St. Petersburg, fish life, water birds, grass beds and shoreline vegetation were much more abundant than they are today. In 1837, John Lee Williams, writing in "The Territory of Florida", wrote that

> "Extensive schools of fish are so thick as to almost impede the boat in shoal waters. Sea fowl are also exceedingly numerous. The beautiful flamingos

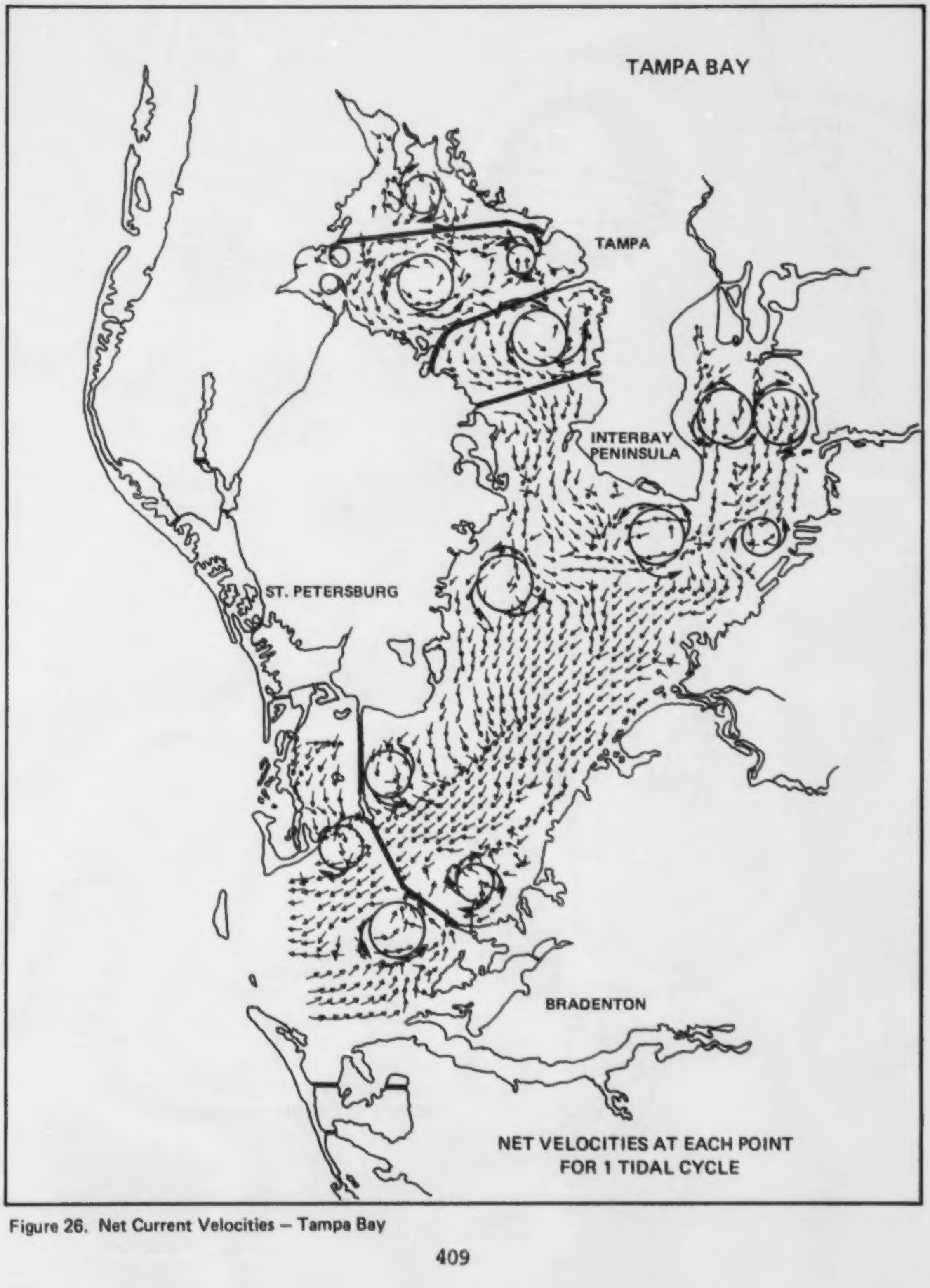

TAMPA BAY
TAMPA
INTERBAY PENINSULA
ST. PETERSBURG
BRADENTON
NET VELOCITIES AT EACH POINT FOR 1 TIDAL CYCLE

Figure 26. Net Current Velocities – Tampa Bay

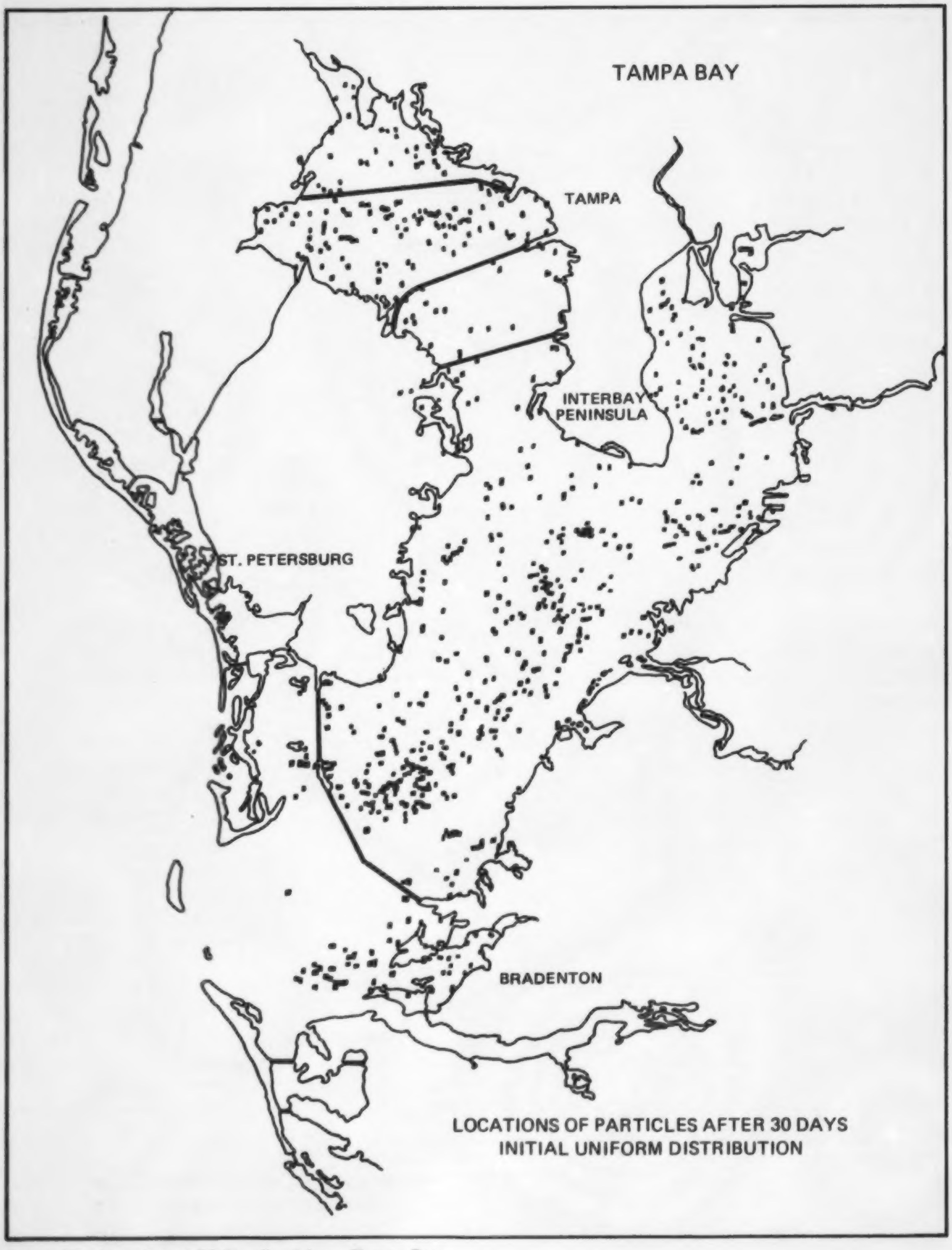

TAMPA BAY
TAMPA
INTERBAY PENINSULA
ST. PETERSBURG
BRADENTON
LOCATIONS OF PARTICLES AFTER 30 DAYS
INITIAL UNIFORM DISTRIBUTION

Figure 27. Location of 30 Day Particles – Tampa Bay

> in particular appear in long
> files."

While it is difficult to quantify statements such as the above, conclusions can be drawn that before human population pressures were present in the area, Tampa Bay was flush with numerous species of fish, fowl and plants. Today, fishing has decreased and fowls and plant life are not as plentiful. Brown pelicans almost disappeared from the area in the late 1960's. Later, the cause for this became known. Pesticides built up in the birds, causing the eggshells to become exceedingly thin and to crack and consequently causing a low hatch rate, depleting the species. After bans on the use of certain pesticides, this species began to populate the area again, albeit somewhat slowly.

Pollution followed man into the Bay areas. Some problems were noted at infrequent intervals in the 1920's. As the area rapidly expanded in the 1950's and 1960's, the problems became intensified. As related to water quality, these problems were manifested as bad odors from the water, high turbidity, high coliforms, low dissolved oxygen and algal growth. The cause of these problems cannot be totally attributed to municipal sewage even though it has a significant part in causing the problems. Industrial discharges, non-point source runoff, residential and urban development and dredging and filling are other causes which had to be corrected in conjunction with treatment of domestic sewage. These all seemed to act in a synergistic manner in causing the above problems and all had to be attacked so solve water quality problems.

The cause of odors starting in the 1920's was found to be the red algae Gracilaria and was found and identified by the Hagan investigative team in 1967. The cause was attributed to an unbalanced ecosystem which had too much carbon, nitrogen and phosphorus materials in the water column. The same was true of the sediment. With this imbalance, the algae grew exceedingly well in the saltwater. Not being tolerant to freshwater which periodically diluted the saltwater, the algae died and the resultant odors were repugnant. This algae generally grew in less than six feet of water. The littoral zone of parts of Tampa Bay received reduced light penetration due in part to increased algae from an overenriched water column, turbidity from dredging and natural color. This reduced light penetration resulted in destruction of saltwater marshes and grasses. The dredging and filling in the 1960's also destroyed much shoreline vegetation. Thus, the problems exacerbated each other and solving one problem did not solve the water quality problem.

Dredging and filling sections of the Bay and shoreline seem to be the most insidious long-term water quality problem in Florida's pollution control efforts. Port development, shoreline alteration and causeways for highways are examples of such dredge and fill operations. Non-point sources followed by domestic and industrial waste discharges appear to be the next worst causes of pollution affecting water quality. The most easily solved problems were point source ones. The point source abatement strategy in Florida was followed two years later with an abatement strategy for dredge and fill operations. Almost nine years after implementation of the above abatement strategies, the regulatory agencies are still seeking the best control stormwater runoff abatement strategy.

Point sources were identified on numerous occasions over the years as being the cause of pollution. This reinforced the point source abatement strategy. Hillsborough Bay was in the worst condition of the bays composing the Tampa Bay system and the volume of point source discharges into that Bay gave credence to the point source strategy. The following tables show the relative contributions of industries and domestic sewage discharges to Hillsborough Bay. (See Figures 28, 29, 30 for BOD, Ammonia and Kjeldahl nitrogen and phosphorus contributions.)

During the 1970's, major reductions were made on phosphorus loadings coming from industrial plants. No water quality studies preceded these abatement efforts and the efforts were based solely on technology-based standards. The effectiveness of this action on industrial plants can be seen by looking at several figures for phosphorus water quality data in the Tampa Bay system monitored during the 1970's. (See Figures 31, 32, 33, 34, 35.) Thus, it is known that industries were the major contributors of high phosphorus levels in the system and that the reduction of phosphorus in the water column was caused by the treatment of the industry's wastewater.

SEWAGE TREATMENT

Confronted with a State law that required advanced sewage treatment without regard to water quality, many of the cities became inventive in order to avoid the expensive capital and operations cost of these requirements. In several instances, cities in the area chose other less expensive means of treatment and effluent disposal or chose to try technology not widely tried. (Figure 36 depicts the location of major municipal sewage plants in the Tampa Bay area.)

Each municipality chose a somewhat different method of addressing the requirements. The following seven cases illustrate the direction each city chose, the design flow of the new facilities, the method of effluent disposal and, in some cases, the schematic of the plant's design. These different approaches were encouraged by a regulatory attitude in Florida that required governments to meet standards without telling them how to do it. It should also be noted that the regulatory scheme is highly decentralized in Florida to the district and county level

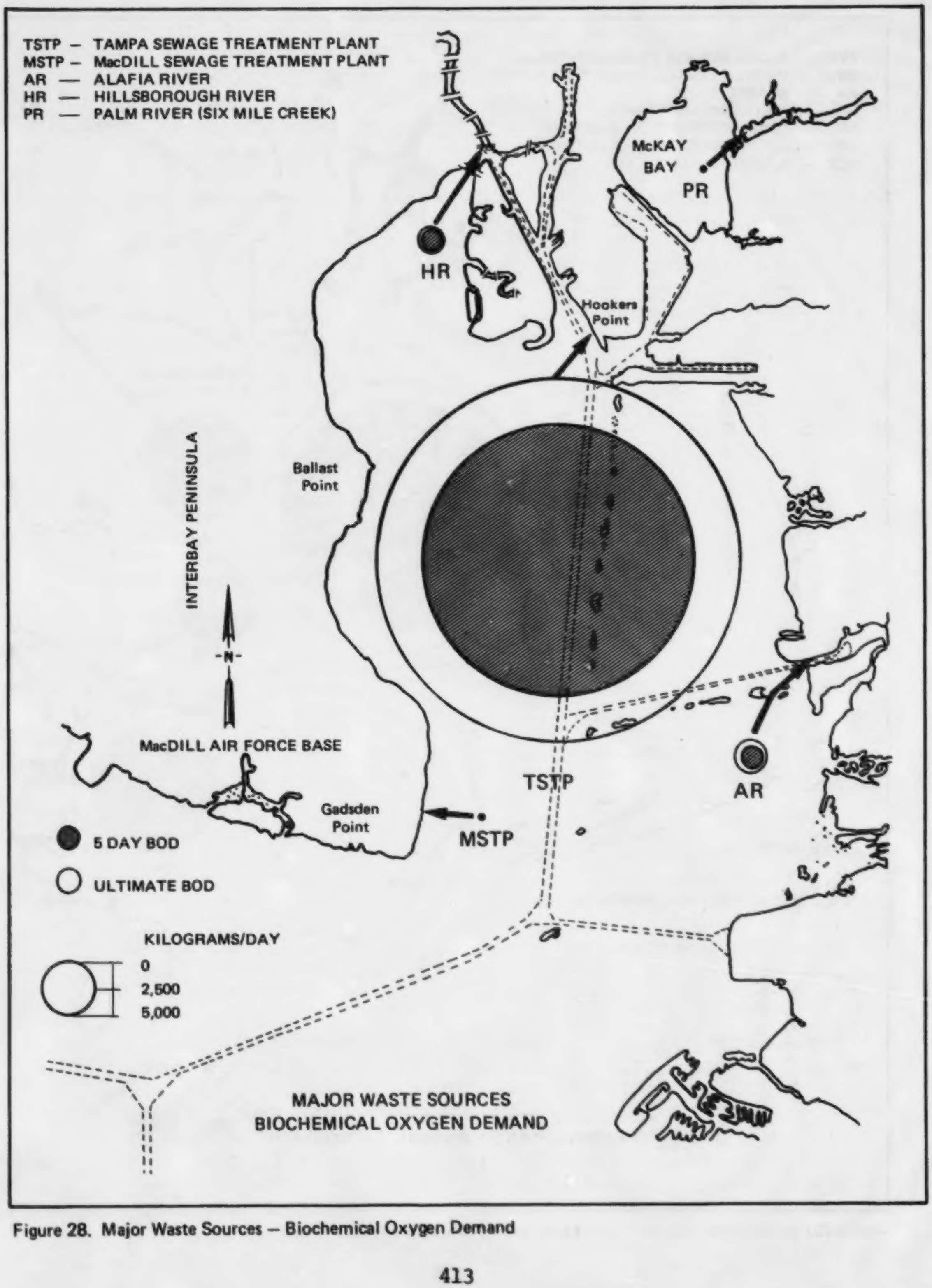

TSTP – TAMPA SEWAGE TREATMENT PLANT
MSTP – MacDILL SEWAGE TREATMENT PLANT
AR — ALAFIA RIVER
HR — HILLSBOROUGH RIVER
PR — PALM RIVER (SIX MILE CREEK)
McKAY BAY
PR
HR
Hookers Point
Ballast Point
INTERBAY PENINSULA
N
MacDILL AIR FORCE BASE
Gadsden Point
MSTP
TSTP
AR
5 DAY BOD
ULTIMATE BOD
KILOGRAMS/DAY
0
2,500
5,000
MAJOR WASTE SOURCES
BIOCHEMICAL OXYGEN DEMAND

Figure 28. Major Waste Sources – Biochemical Oxygen Demand

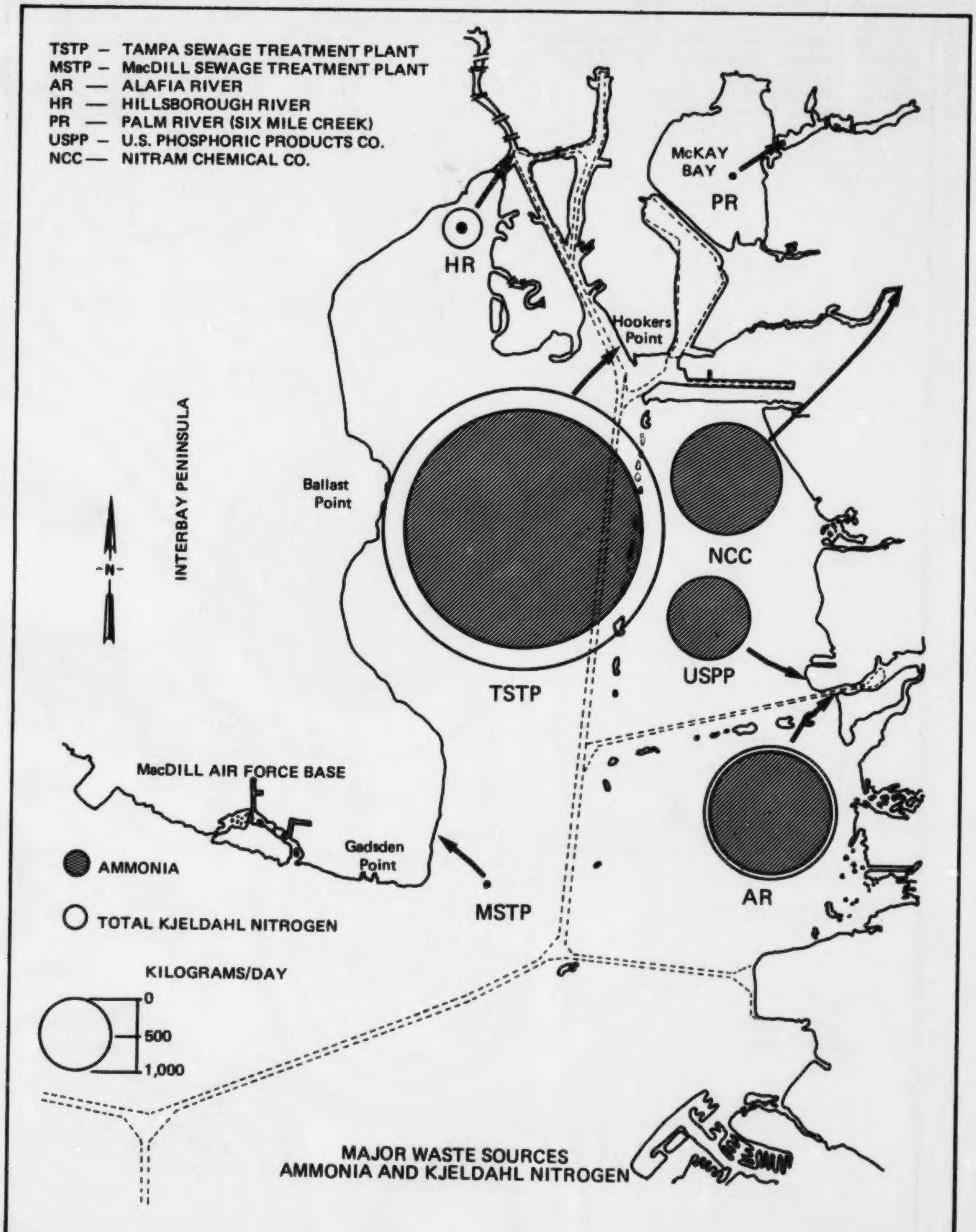

TSTP – TAMPA SEWAGE TREATMENT PLANT
MSTP – MacDILL SEWAGE TREATMENT PLANT
AR — ALAFIA RIVER
HR — HILLSBOROUGH RIVER
PR — PALM RIVER (SIX MILE CREEK)
USPP – U.S. PHOSPHORIC PRODUCTS CO.
NCC — NITRAM CHEMICAL CO.
McKAY BAY
PR
HR
Hookers Point
INTERBAY PENINSULA
Ballast Point
N
TSTP
NCC
USPP
MacDILL AIR FORCE BASE
Gadsden Point
MSTP
AR
AMMONIA
TOTAL KJELDAHL NITROGEN
KILOGRAMS/DAY
0
500
1,000
MAJOR WASTE SOURCES
AMMONIA AND KJELDAHL NITROGEN

Figure 29. Major Waste Sources – Ammonia and Kjeldahl Nitrogen

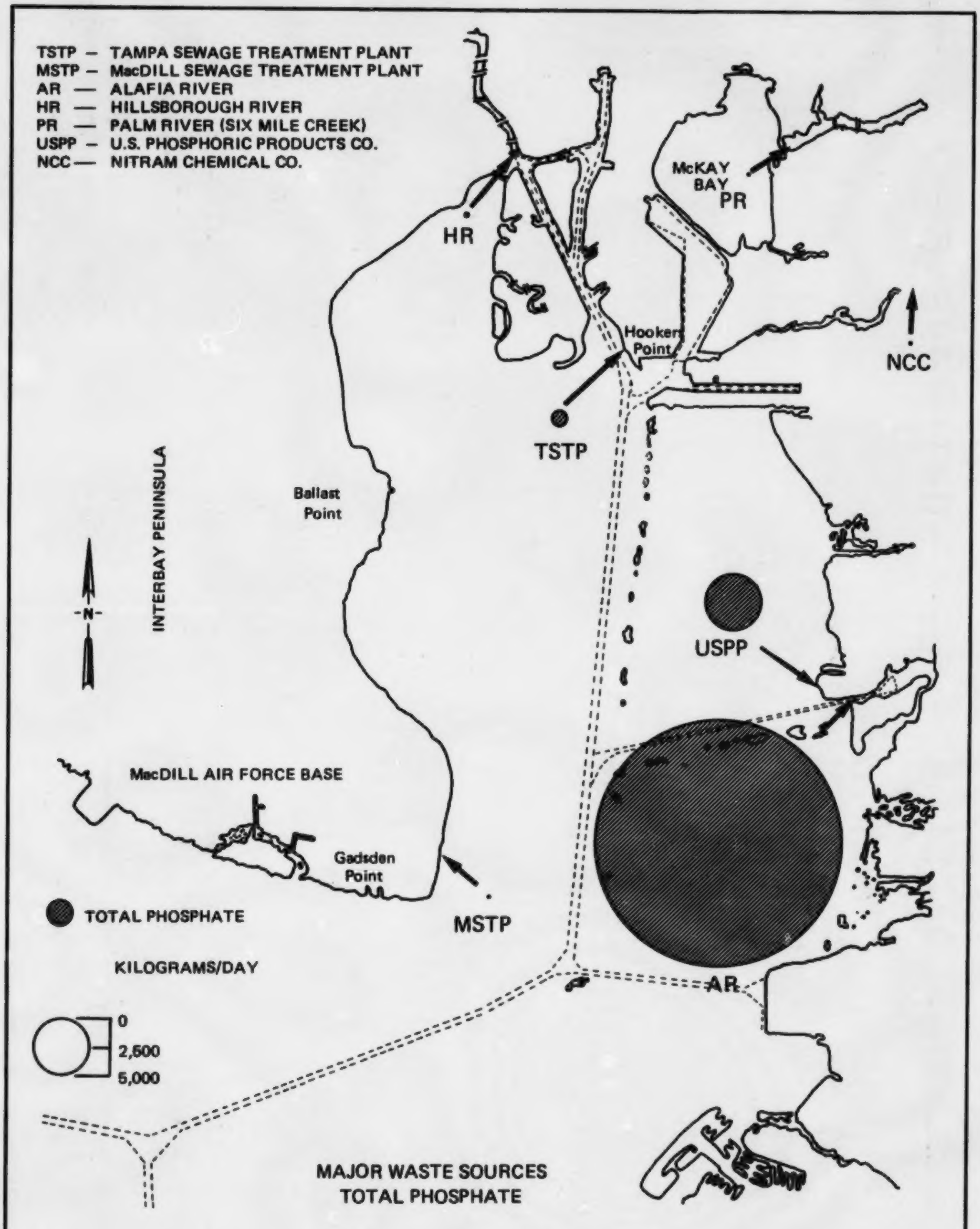

TSTP — TAMPA SEWAGE TREATMENT PLANT
MSTP — MacDILL SEWAGE TREATMENT PLANT
AR — ALAFIA RIVER
HR — HILLSBOROUGH RIVER
PR — PALM RIVER (SIX MILE CREEK)
USPP — U.S. PHOSPHORIC PRODUCTS CO.
NCC — NITRAM CHEMICAL CO.
HR
McKAY BAY
PR
Hooker Point
NCC
TSTP
INTERBAY PENINSULA
Ballast Point
N
USPP
MacDILL AIR FORCE BASE
Gadsden Point
MSTP
TOTAL PHOSPHATE
KILOGRAMS/DAY
0
2,500
5,000
AR
MAJOR WASTE SOURCES
TOTAL PHOSPHATE

Figure 30. Major Waste Sources – Total Phosphate

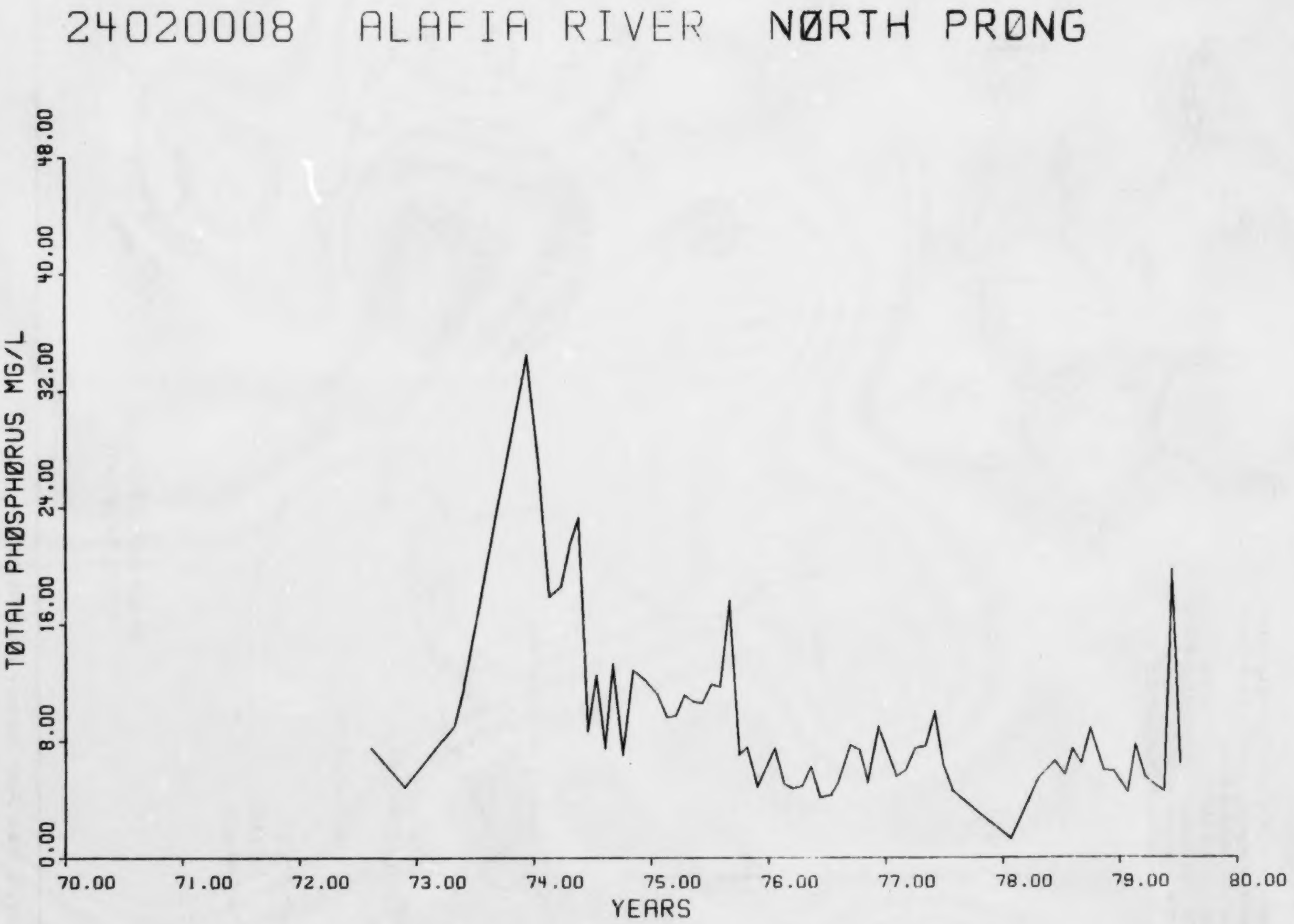


Figure 31. Phosphorus Levels

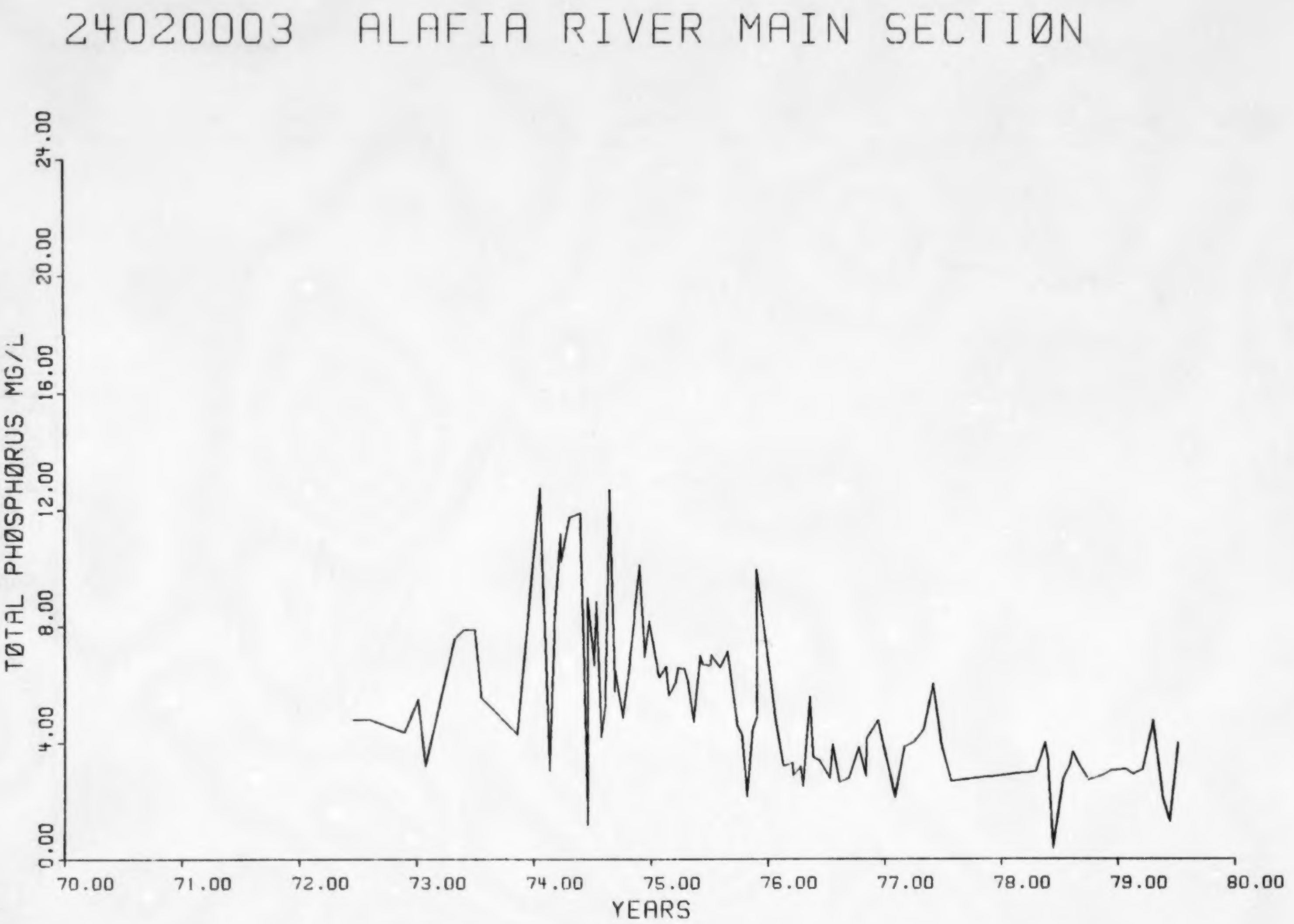


Figure 32. Phosphorus Levels

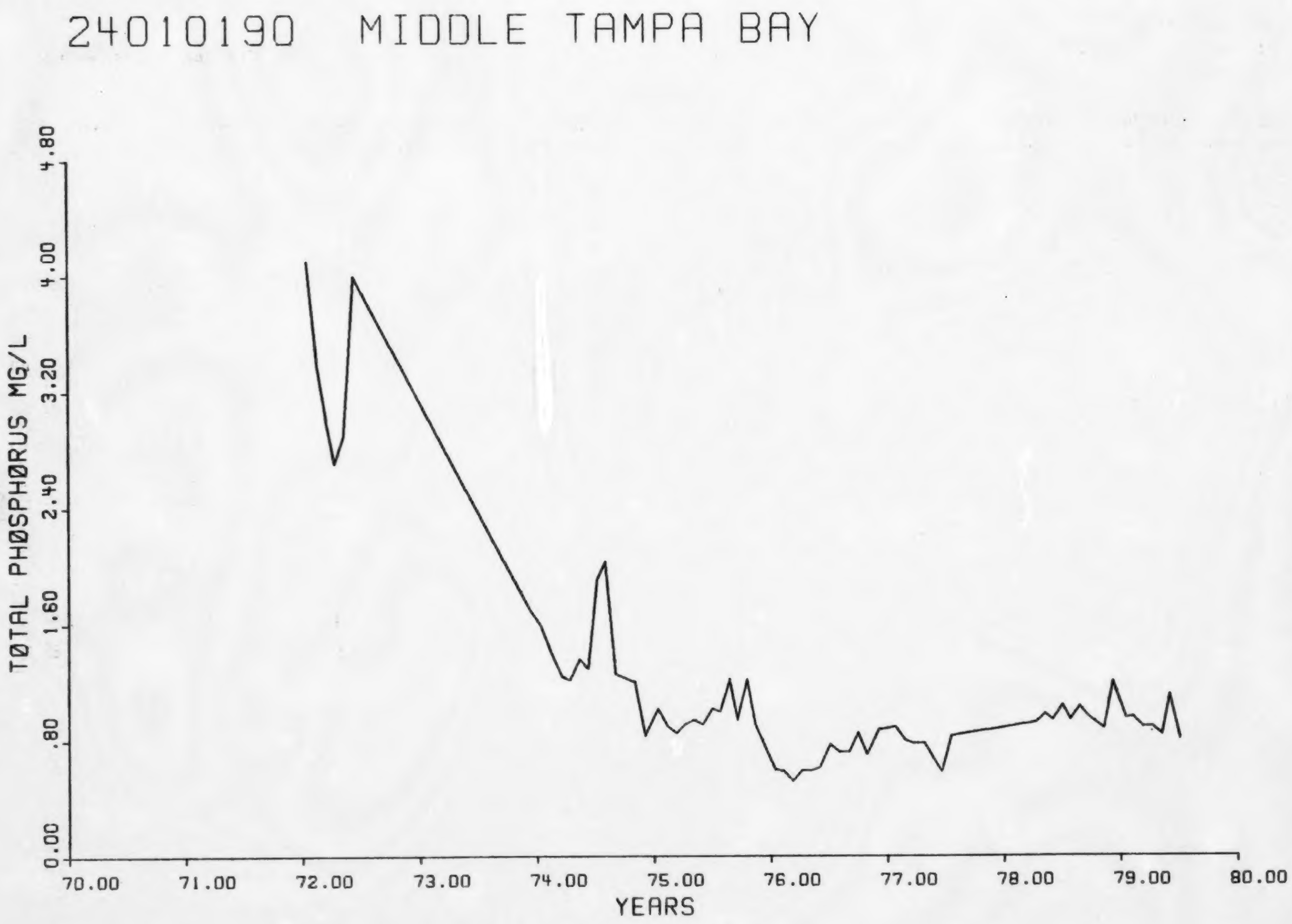


Figure 33. Phosphorus Levels

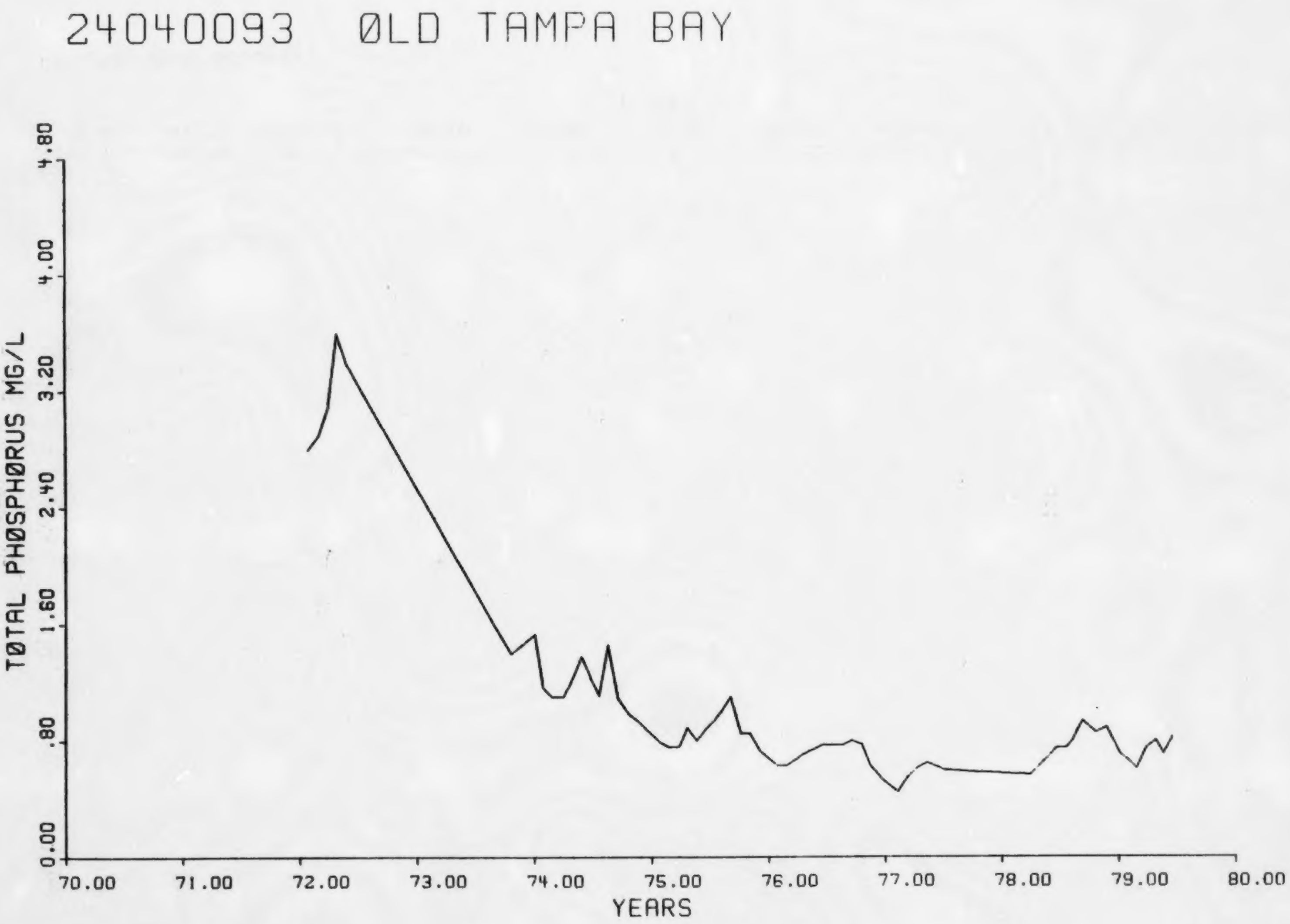


Figure 34. Phosphorus Levels

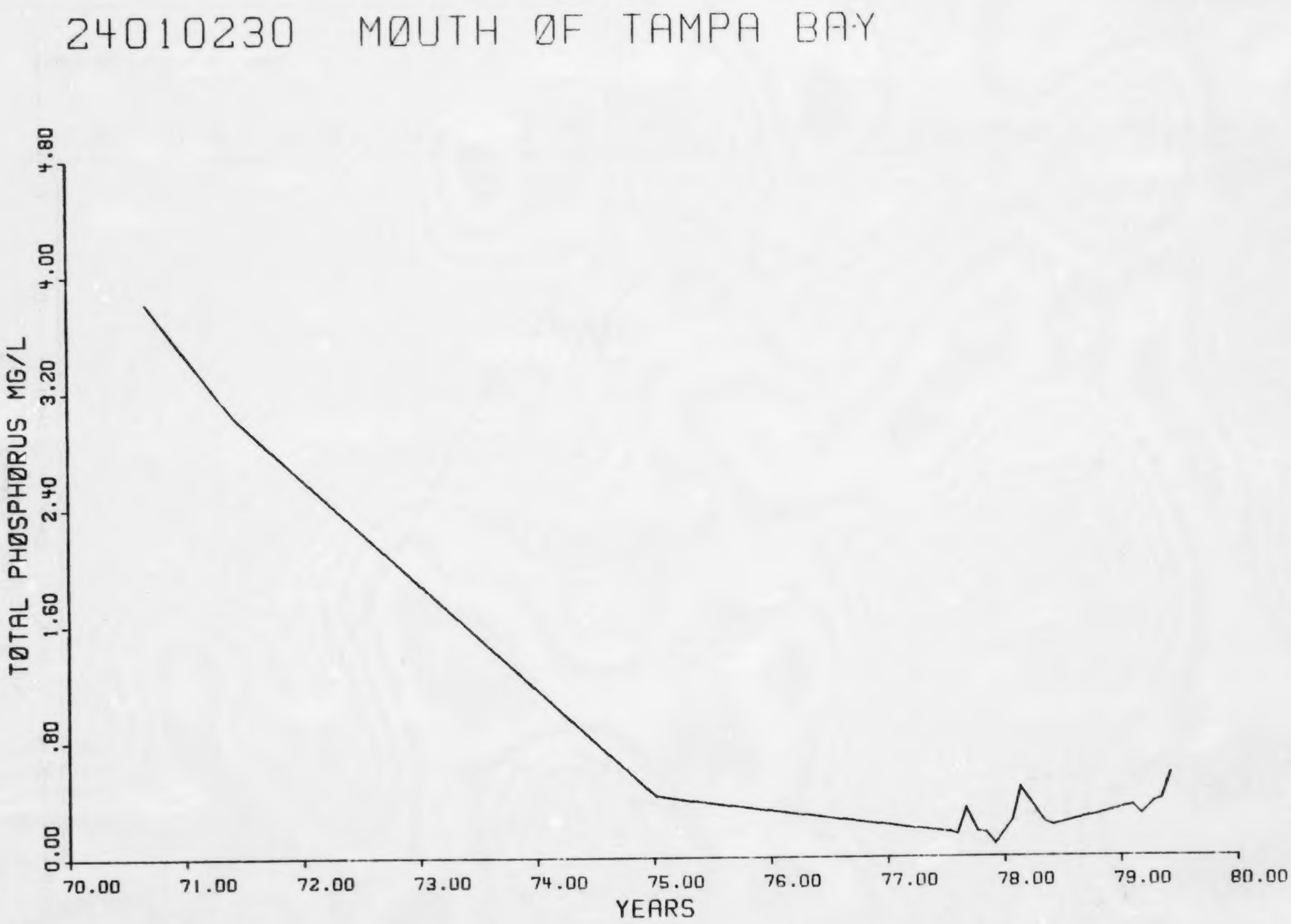


Figure 35. Phosphorus Levels

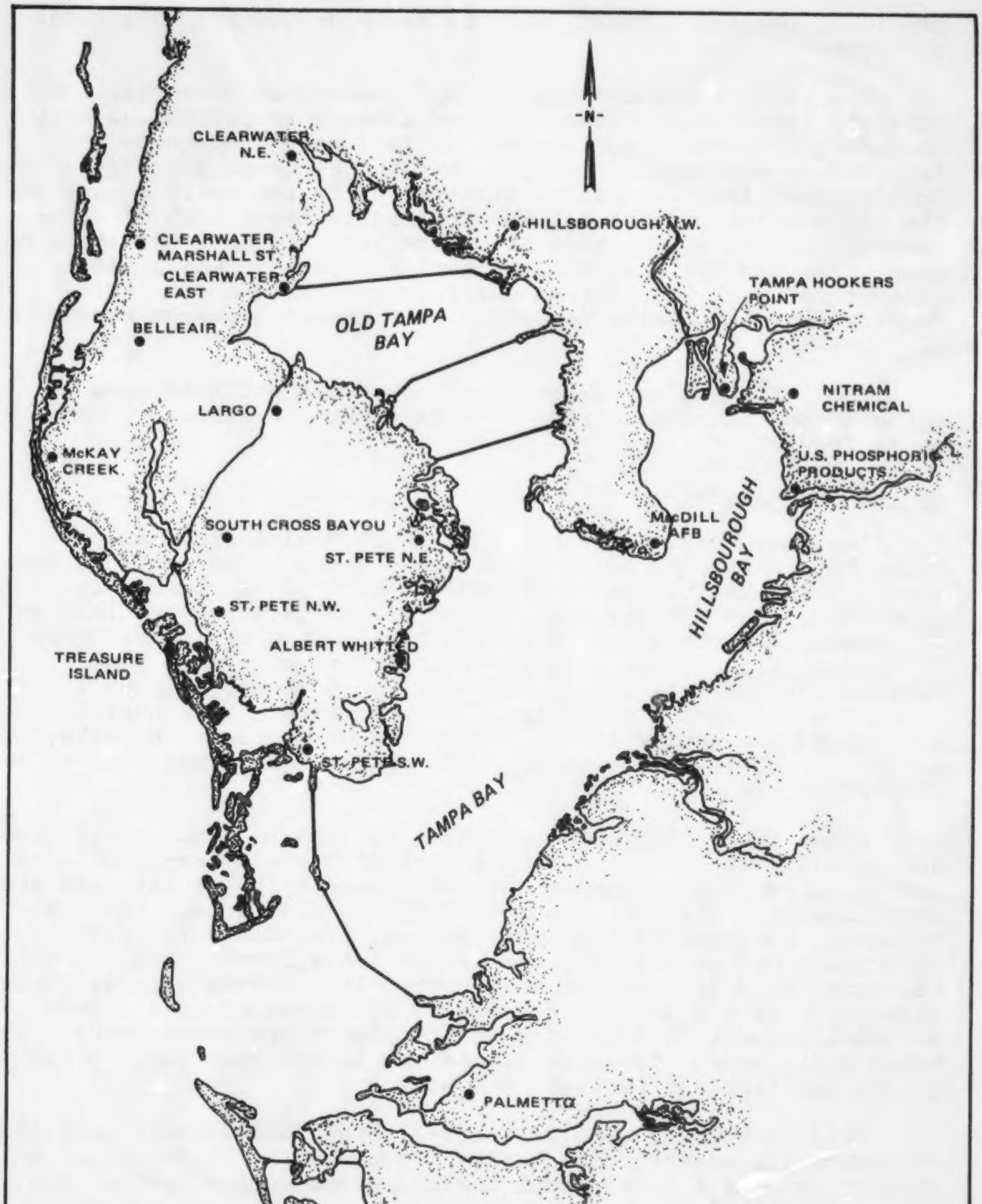

-N-
CLEARWATER N.E.
HILLSBOROUGH N.W.
CLEARWATER MARSHALL ST.
CLEARWATER EAST
TAMPA HOOKERS POINT
OLD TAMPA BAY
BELLEAIR
NITRAM CHEMICAL
LARGO
U.S. PHOSPHORIC PRODUCTS
McKAY CREEK
McDILL AFB
SOUTH CROSS BAYOU
ST. PETE N.E.
HILLSBOUROUGH BAY
ST. PETE N.W.
ALBERT WHITTED
TREASURE ISLAND
ST. PETE S.W.
TAMPA BAY
PALMETTO

Figure 36. Major Municipal Dischargers – Tampa Bay – 1970

providing maximum citizen input to decisions affecting a local government.

The legislation calling for AWT came about as a result of numerous local perceptions. One of these perceptions was that secondary treatment requirements in 1972 were inadequate and legislation was passed requiring AWT in the Tampa Bay area. The geographical area was rather limited during the early stages of the legislative process but was defined to cover much of west-central Florida by the time it became law. Perhaps the lobbying group that had the most influence on its passage was a local district of Girl Scouts that was very concerned about their recreational camp being too close to a proposed sewage treatment plant.

The following are examples of how municipalities have changed the method of disposal or have met the statutory requirements for AWT.

St. Petersburg STP

The City has four plants that in 1970 discharged to the Tampa Bay system and adjacent coastal waters. The combined design flow of the plants in the 1980's will be approximately $2.26X10^5$ m^3/day. Today all but one plant discharges to land on the peninsula. This remaining plant will join the other three in land application upon completion of construction. Land application has been accomplished by two methods, deep well injection and spray irrigation. Deep wells are used during wet periods and spray irrigation during dry periods on parks, golf courses, lawns and other green spaces. Deep wells are 275-400 meters deep.

Since the public is very likely to come in contact with the spray, it has been treated to be virtually virus free. This is accomplished by secondary treatment, coagulation, filtration and chlorination. The City spent $400,000 to have a well known virologist, who opposed the spray system, test for viricidal efficiency before starting the irrigation systems. After demonstrating the efficiency of this system in removing viruses, this virologist is now one of the leading advocates of this system. An added benefit is the water and nutrients are reused in a water short area. Treasure Island, an independent city, opted to tie into the St. Petersburg system.

This is truly a case of federal, state and local cooperation in making the construction grants program work. Today the constraints at the federal level would prevent such a system from coming to fruition.

South Cross Bayou and McKay Creek STP

These two facilities are owned by Pinellas County and the County has opted to remove their surface discharges to deep well injections. These deep wells are 275-400 meters deep with no significant overlying freshwater aquifers. These systems ultimately will displace saltwater far out into the Gulf of Mexico. The total design flow in the 1980's is $1.26X10^5$ m^3/day.

Belleair STP

This municipality is composed of wealthy individuals and they wanted to retain their own system and discharge their effluent to the Gulf of Mexico. Their system was thus required to have AWT with a design flow of $1.89X10^3$ m^3/day. This system is an aeration system for both carboneous and nitrogenous components.

Largo STP

This City had a very innovative engineer who designed an AWT Plant for the cost of a normal secondary system. It was designed for $3.4X10^4$ m^3/day but has never used chemicals in the AWT system. The effluent was filtered, however. Instead it discharged to a series of ponds around a golf course. The water from the ponds was sprayed on the golf course and no discharges occurred except during very rainy periods when the discharge is allowed to drain through a swampy mangrove marsh before entering the Bay system. Today, the City has contracted with a private company to convert part of the existing system to AWT using existing structures. This new experimental process does not use energy expensive chemicals. This is a demonstration project between private industry and a local government costing the local government only a margin of the cost of a regular project. (See Figure 37 for schematic of the retrofitted plant.)

Clearwater STP

This City has been caught and squeezed between changing planning requirements and regulatory inconsistencies. At present, one plant is nitrifing and discharging to a stream, one plant was supposed to go to a deep well system that did not work, one plant discharges to a spray irrigation site with a high groundwater table and one plant discharges to Tampa Bay. All systems are waiting for the completion of an Environmental Impact Statement by the U.S. EPA. The sum total of design flows for these facilities is $6.42X10^4$ m^3/day.

Hillsborough N.W. STP

This is a county owned plant that has been forbidden to expand because of water quality constraints. It has a design capacity of $1.7X10^4$ m^3/day and provides a high quality AWT

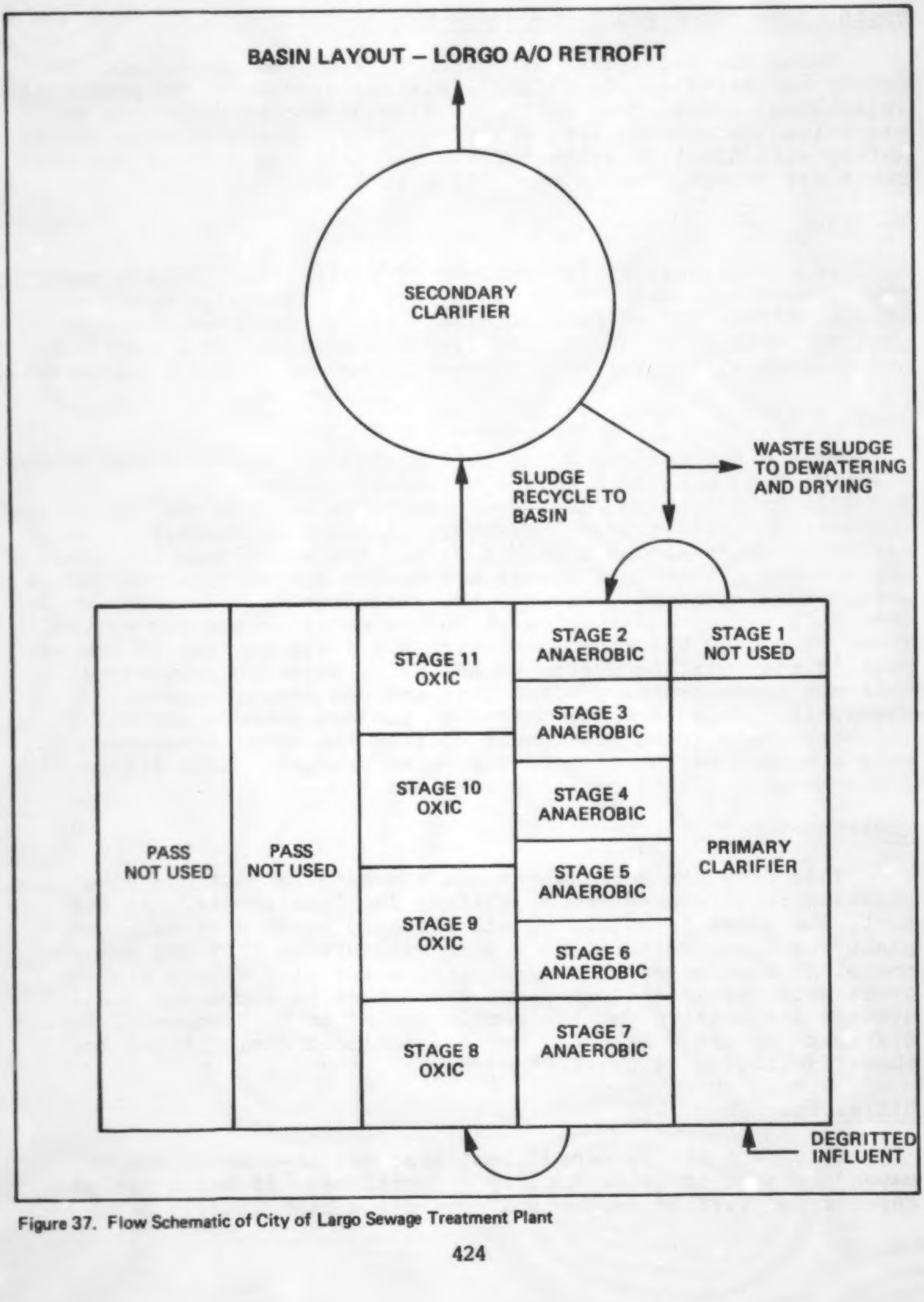

BASIN LAYOUT – LORGO A/O RETROFIT
SECONDARY CLARIFIER
WASTE SLUDGE TO DEWATERING AND DRYING
SLUDGE RECYCLE TO BASIN
PASS NOT USED
PASS NOT USED
STAGE 11 OXIC
STAGE 10 OXIC
STAGE 9 OXIC
STAGE 8 OXIC
STAGE 2 ANAEROBIC
STAGE 3 ANAEROBIC
STAGE 4 ANAEROBIC
STAGE 5 ANAEROBIC
STAGE 6 ANAEROBIC
STAGE 7 ANAEROBIC
STAGE 1 NOT USED
PRIMARY CLARIFIER
DEGRITTED INFLUENT

Figure 37. Flow Schematic of City of Largo Sewage Treatment Plant

effluent that meets the restrictive AWT standards. (See Figure 38 for schematic of the plant.)

Tampa

This City perhaps has the largest AWT Plant in the United States. It has a design flow of $2.26X10^5$ m^3/day and is highly complex. In the past, this discharge has been attributed to be the cause of many of the water quality problems in Hillsborough Bay. There is little question that it is a contributor, the extent of its contribution is unknown. The plant is today achieving effluent AWT limitations that meet the very strict standards set ten years ago. (See Figure 39 for the schematic of the plant.)

Palmetto

In contrast with most other cities in the State of Florida, this City decided receipt of federal dollars did not justify the commensurate "red tape" (excessive requirements and paperwork). The City built an innovative sewage treatment plant, solely with their own money, that would meet the State's requirements. This plant's processes are generally known as the "Bardenpho Process" and is not chemically nor overly energy intensive.

These are some of the major municipal facilities that encircle Tampa Bay. Their reaction to the requirements of AWT mandated by statute and defined by regulation are informative. To this day, the Tampa Bay system still has some water quality problems. There are some indications of recovery though only time will tell if this statutory mandate expedited the clean up.

CHEMICAL AND BIOLOGICAL WATER QUALITY

Many low values of DO have been recorded during the summer months for several years. BOD levels have remained fairly low even though large quantities were discharged into Hillsborough Bay. During periods of high odor intensity in 1967, however, the DO was never less than 5.1 mg/l. Therefore, it was concluded that the odor problem was not a direct function of soluble and suspended organic material depleting DO through anaerobic decomposition.

During the intensive studies of 1968, the chlorophyll a mean concentration was determined to be 35 mg/l indicating a highly tropic level with a large plant biomass caused by an excess of nutrients. Figure 40 shows the levels of chlorophyll a found in the bay system in 1978. Data for 1979 reveal that the chlorophyll a still reaches 35-40 mg/l, although the mean in the worst polluted areas is decreasing.

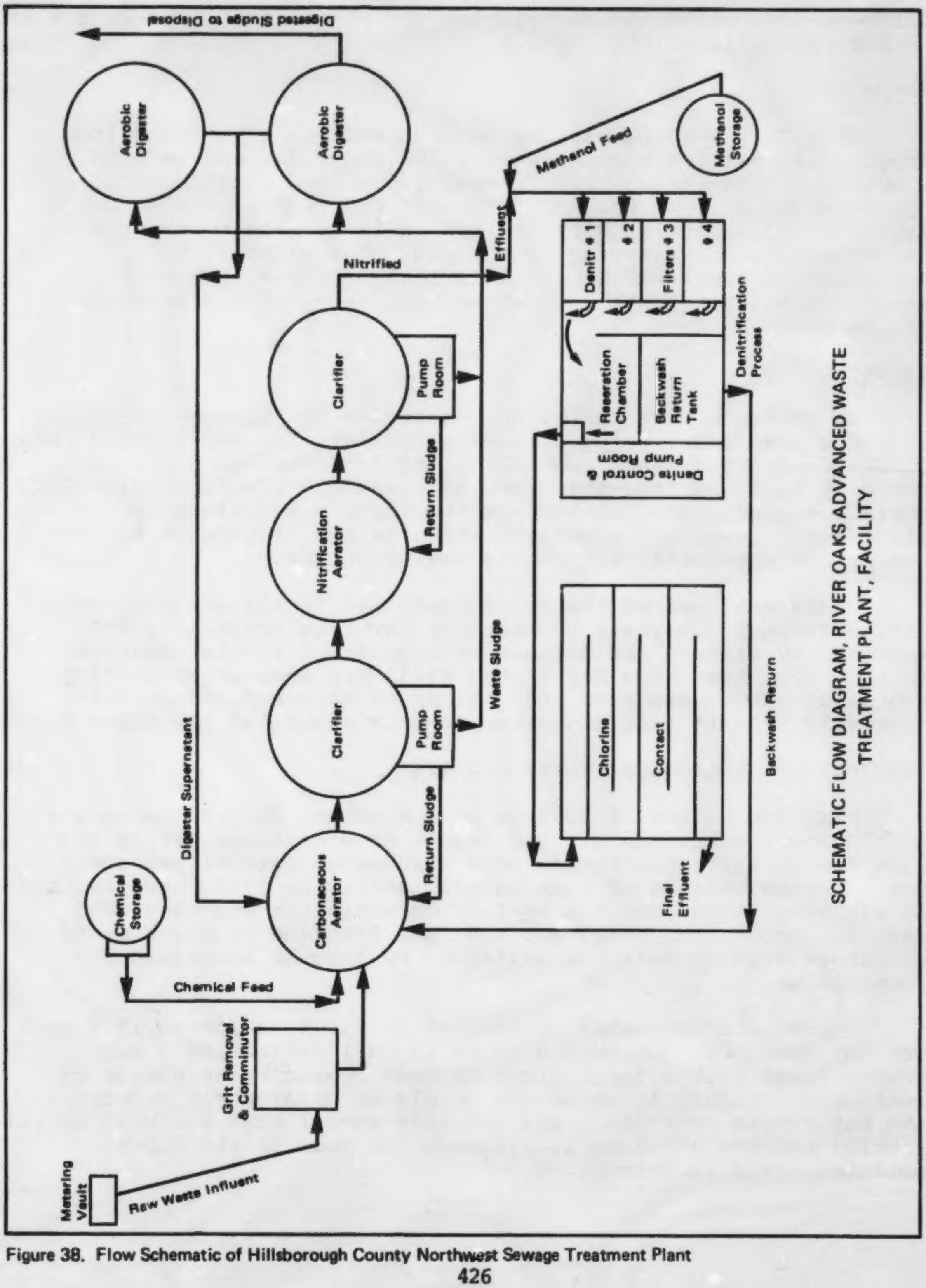

SCHEMATIC FLOW DIAGRAM, RIVER OAKS ADVANCED WASTE TREATMENT PLANT, FACILITY
Metering Vault
Raw Waste Influent
Grit Removal & Comminutor
Chemical Storage
Chemical Feed
Carbonaceous Aerator
Digester Supernatant
Return Sludge
Clarifier
Pump Room
Waste Sludge
Nitrification Aerator
Return Sludge
Clarifier
Pump Room
Nitrified
Effluent
Aerobic Digester
Aerobic Digester
Digested Sludge to Disposal
Methanol Feed
Methanol Storage
Denitr # 1
# 2
Filters # 3
# 4
Reaeration Chamber
Backwash Return Tank
Denite Control & Pump Room
Denitrification Process
Chlorine
Contact
Final Effluent
Backwash Return

Figure 38. Flow Schematic of Hillsborough County Northwest Sewage Treatment Plant

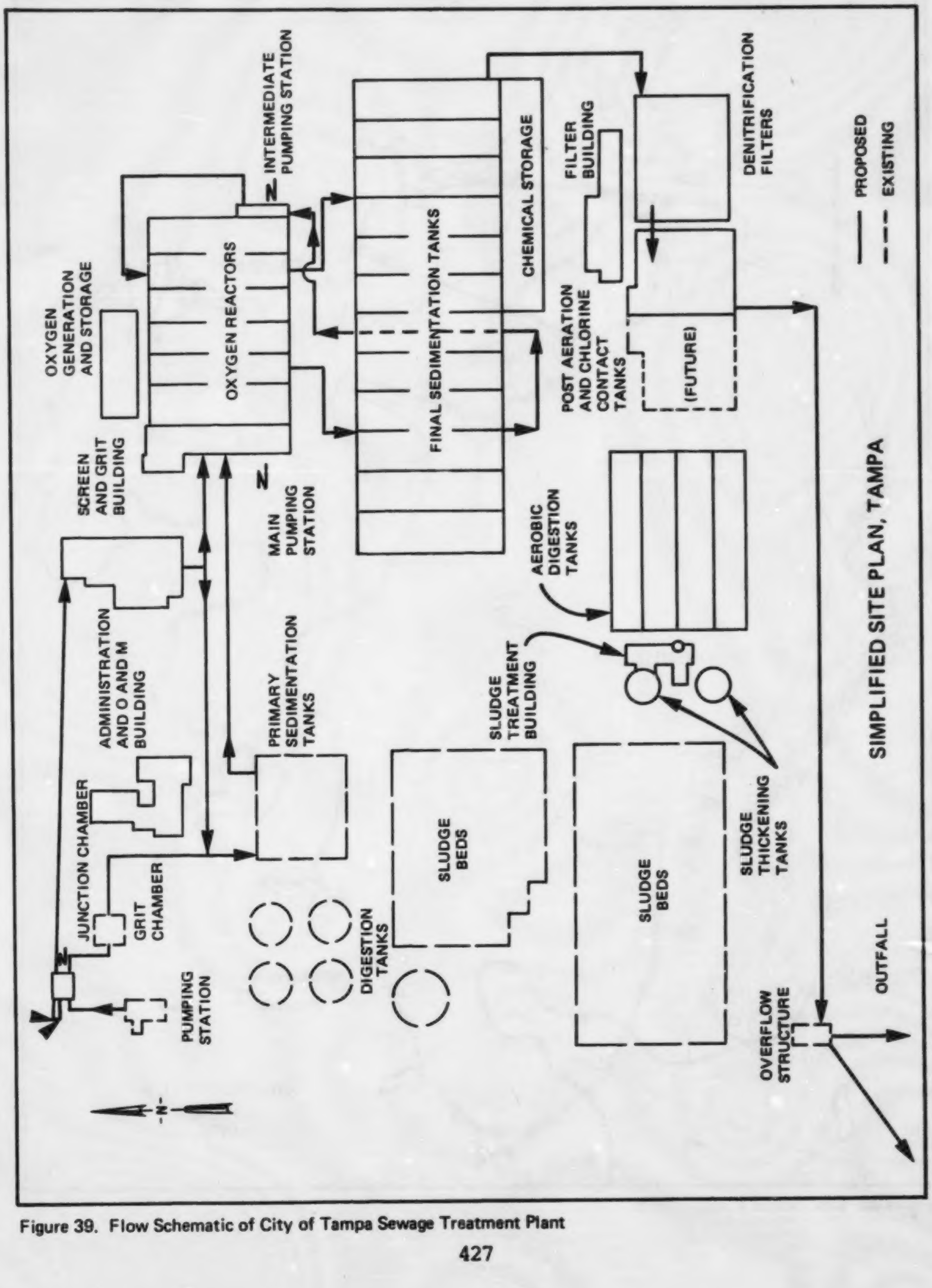

SIMPLIFIED SITE PLAN, TAMPA
N
PUMPING STATION
JUNCTION CHAMBER
GRIT CHAMBER
DIGESTION TANKS
ADMINISTRATION AND O AND M BUILDING
PRIMARY SEDIMENTATION TANKS
SCREEN AND GRIT BUILDING
MAIN PUMPING STATION
OXYGEN GENERATION AND STORAGE
OXYGEN REACTORS
INTERMEDIATE PUMPING STATION
FINAL SEDIMENTATION TANKS
CHEMICAL STORAGE
SLUDGE BEDS
SLUDGE TREATMENT BUILDING
AEROBIC DIGESTION TANKS
POST AERATION AND CHLORINE CONTACT TANKS
(FUTURE)
FILTER BUILDING
DENITRIFICATION FILTERS
SLUDGE BEDS
SLUDGE THICKENING TANKS
OVERFLOW STRUCTURE
OUTFALL
PROPOSED
EXISTING

Figure 39. Flow Schematic of City of Tampa Sewage Treatment Plant

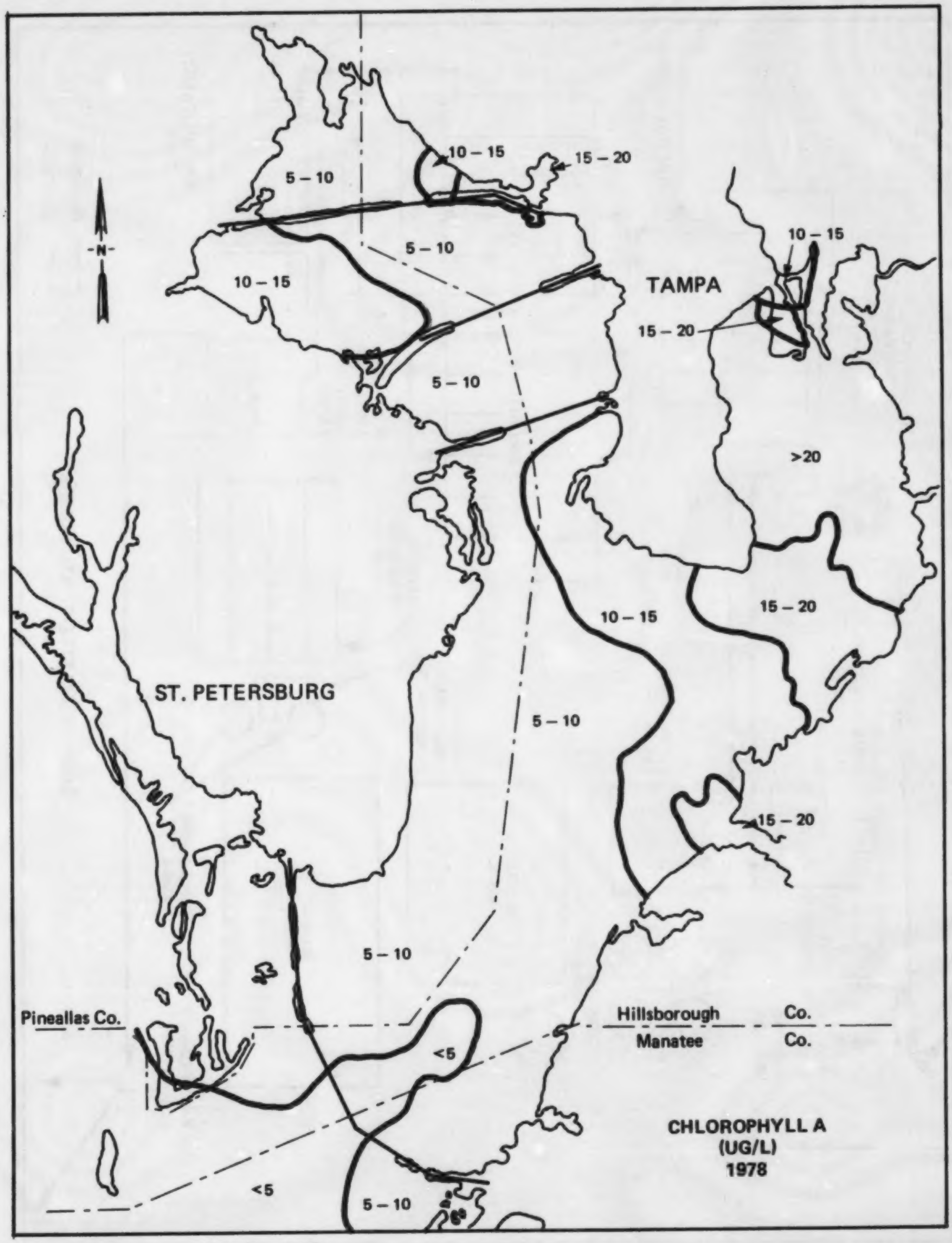

10 – 15
15 – 20
5 – 10
-N-
5 – 10
10 – 15
TAMPA
10 – 15
15 – 20
5 – 10
>20
10 – 15
15 – 20
ST. PETERSBURG
5 – 10
15 – 20
5 – 10
Pineallas Co.
Hillsborough Co.
Manatee Co.
<5
CHLOROPHYLL A
(UG/L)
1978
<5
5 – 10

Figure 40. Chlorophyll A ug/l – 1978

Limiting nutrient algal assay studies conducted in 1975 by FDER revealed that in all cases the nitrogen spiked sample waters produced algal growth greater than either the control or phosphorus spiked samples. These results indicate that at all stations and dates sampled, nitrogen was the nutrient limiting phytoplankton growth. (See Figures 41, 42 for results of these tests as well as sample location.)

The City of Tampa in 1979 requested a variance from FDER seeking to discontinue phosphorus removal. Based on the algal assay studies and previous industrial phosphorus removal, FDER granted the request. This saved the City considerable money and there was evidence that water quality was not being enhanced by this requirement. Nitrogen, however, is still required to be removed.

It was mentioned earlier in this paper that phosphorus levels in the water column had decreased over the last ten years. In most cases, the level has dropped two to ten fold over this time frame. Figure 43 shows that most of the phosphorus problem in the Bay is caused by the Alafia River. When the loading in this river decreased, the level in the Bay decreased.

Historical data shows the Tampa Sewage Plant to be a major cause of ammonia and coliform pollution. Figure 44 clearly shows the ammonia levels in Hillsborough Bay were caused by the Tampa Sewage Treatment Plant. Fecal coliform bacteria in Hillsborough Bay may not be solely derived from the Tampa Sewage Plant but isopletes done in 1968 show a large number are caused by this plant. (See Figure 45.)

Extensive monitoring in Tampa Bay has been conducted since 1972 by both Hillsborough County and FDER. Figure 46 depicts the location of 54 weekly and/or monthly sampling locations in the Bay conducted by Hillsborough County. Numerous parameters are analyzed. Hillsborough County 1978 data confirm that significant coliform reductions occurred after the Tampa Plant went into operation. (See Figure 47.) FDER data for 1979 show significant reductions in TKN and fecal coliform. (See Figures 48, 49 for comparison of Recent (R) versus Historical (H) pollutant levels in both arms of Tampa Bay.

Gracilaria, the red algae in Hillsborough Bay, is present because of the excess nutrients. Its excessive volume is killed off in the presence of freshwater thus causing a bad odor problem. It is hoped that reductions in the nitrogen and phosphorus will eventually clean up this problem in the Tampa Bay system.

Biological data collected by FDER over the last seven to eight years for macroinvertebrates show some improvements as shown in Figures 50, 51, 52, 53. Hillsborough Bay in particular appears to be showing some improvement. A comprehensive

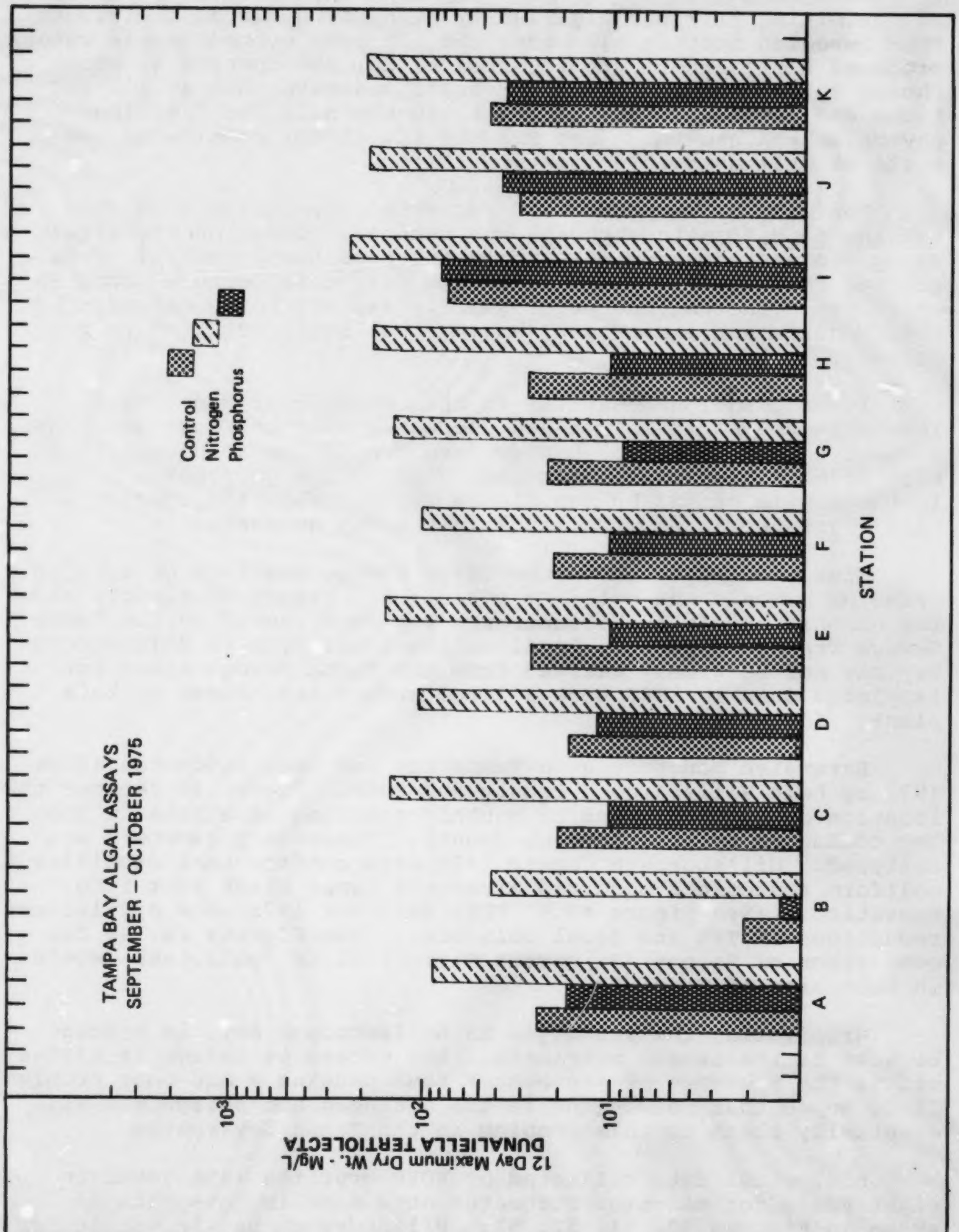


Figure 41. Tampa Bay Algal Assays – September-October, 1975

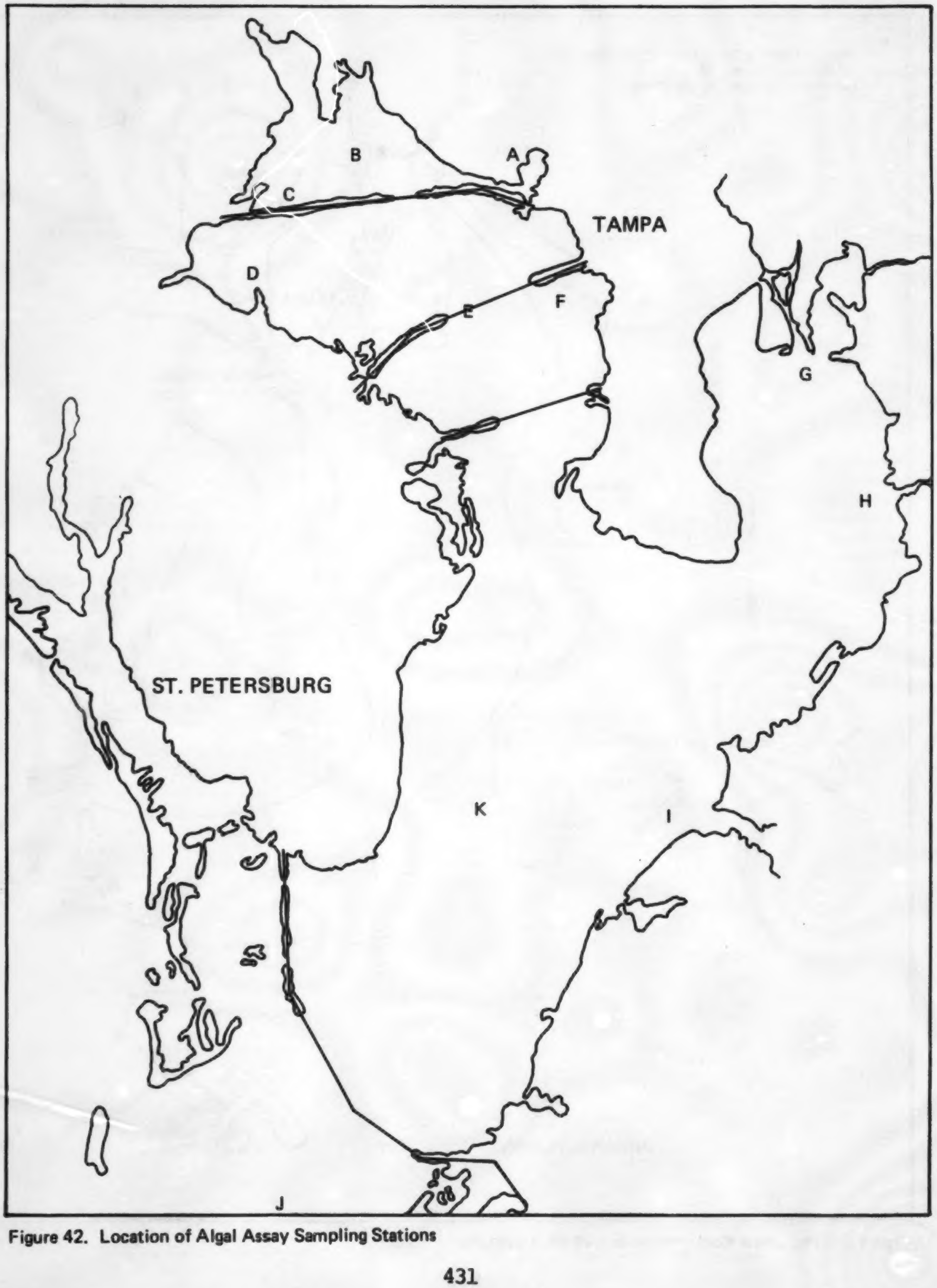

B
A
C
TAMPA
D
E
F
G
H
ST. PETERSBURG
K
I
J

Figure 42. Location of Algal Assay Sampling Stations

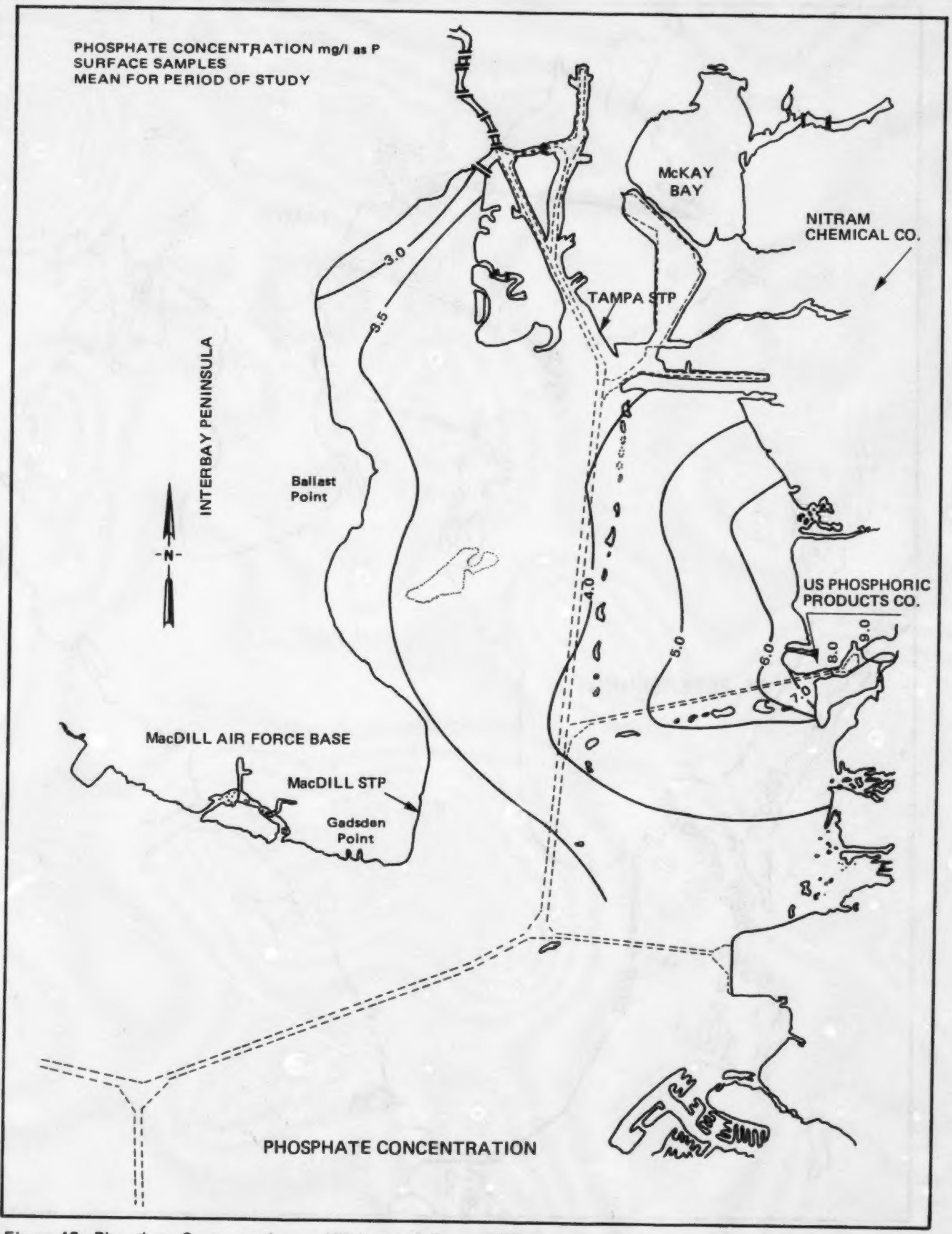

PHOSPHATE CONCENTRATION mg/l as P
SURFACE SAMPLES
MEAN FOR PERIOD OF STUDY
McKAY BAY
NITRAM CHEMICAL CO.
3.0
3.5
TAMPA STP
INTERBAY PENINSULA
Ballast Point
N
4.0
5.0
6.0
7.0
8.0
9.0
US PHOSPHORIC PRODUCTS CO.
MacDILL AIR FORCE BASE
MacDILL STP
Gadsden Point
PHOSPHATE CONCENTRATION

Figure 43. Phosphate Concentrations – Hillsborough Bay – 1969

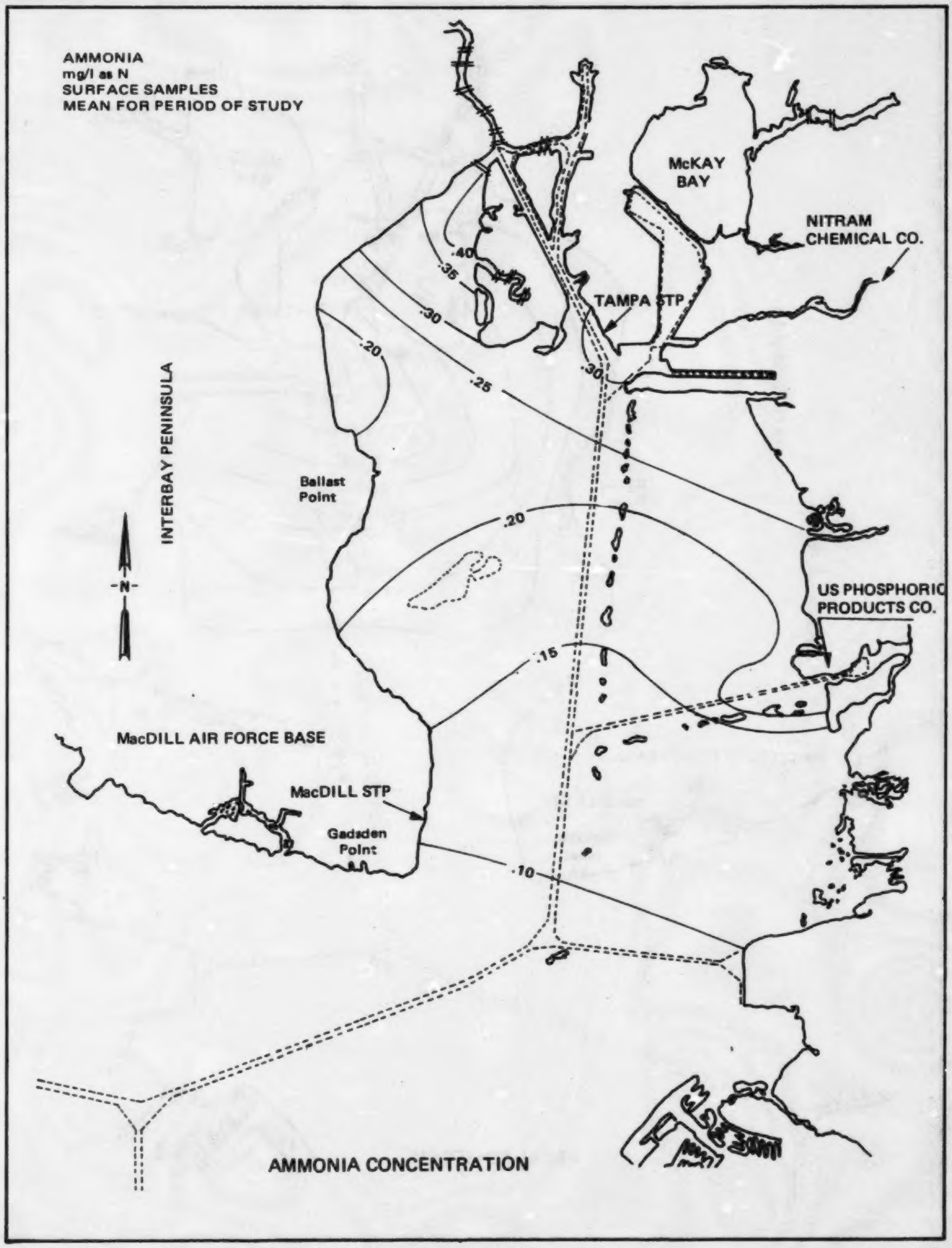

AMMONIA
mg/l as N
SURFACE SAMPLES
MEAN FOR PERIOD OF STUDY
McKAY BAY
NITRAM CHEMICAL CO.
TAMPA STP
.40
.35
.30
.20
.25
.30
INTERBAY PENINSULA
-N-
Ballast Point
.20
US PHOSPHORIC PRODUCTS CO.
.15
MacDILL AIR FORCE BASE
MacDILL STP
Gadsden Point
.10
AMMONIA CONCENTRATION

Figure 44. Ammonia Concentrations – Hillsborough Bay – 1969

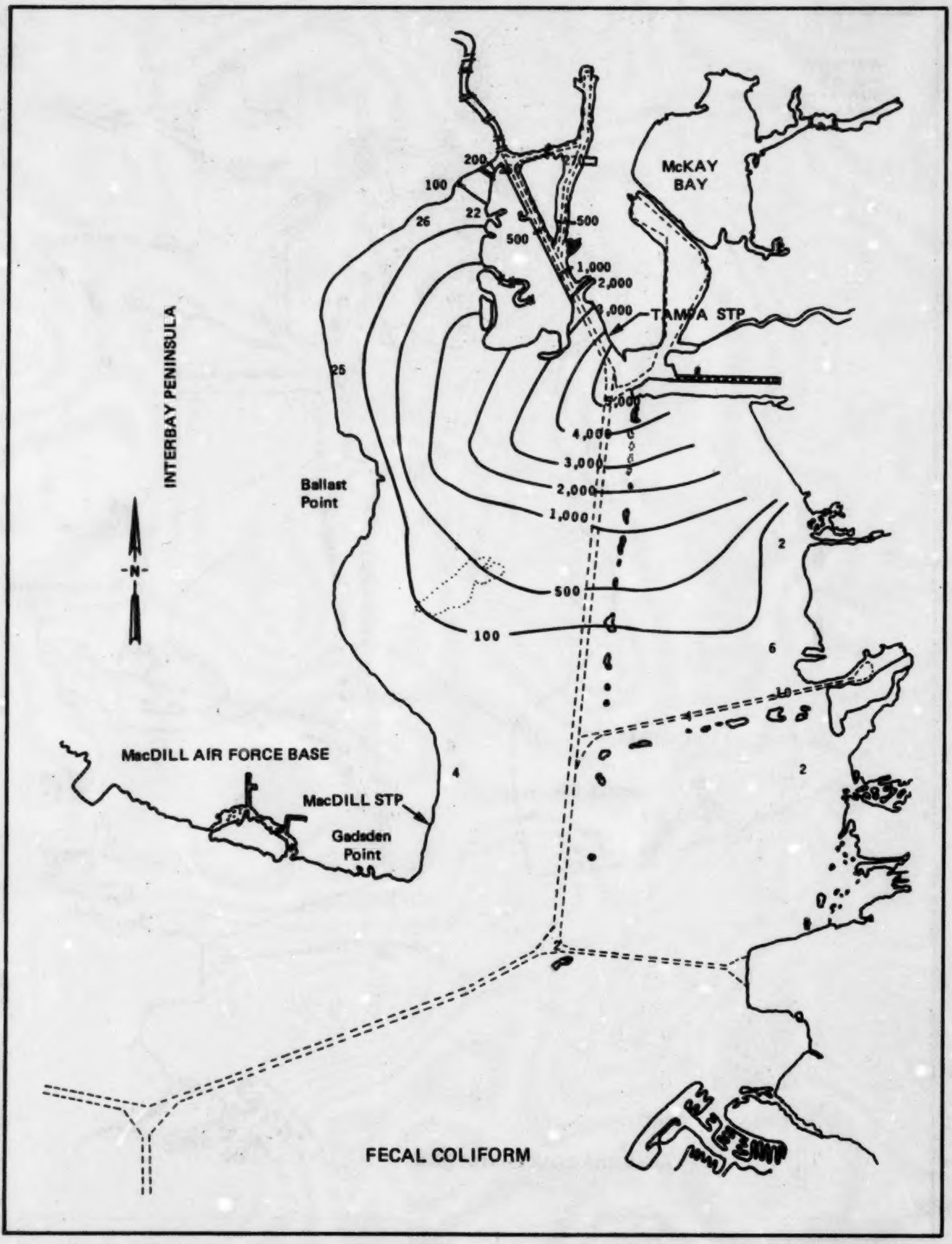

McKAY BAY
200
100
22
26
500
500
1,000
2,000
3,000
TAMPA STP
INTERBAY PENINSULA
25
5,000
4,000
3,000
2,000
1,000
500
100
Ballast Point
N
2
6
10
2
4
MacDILL AIR FORCE BASE
MacDILL STP
Gadsden Point
FECAL COLIFORM

Figure 45. Fecal Coliform – Hillsborough Bay – 1969

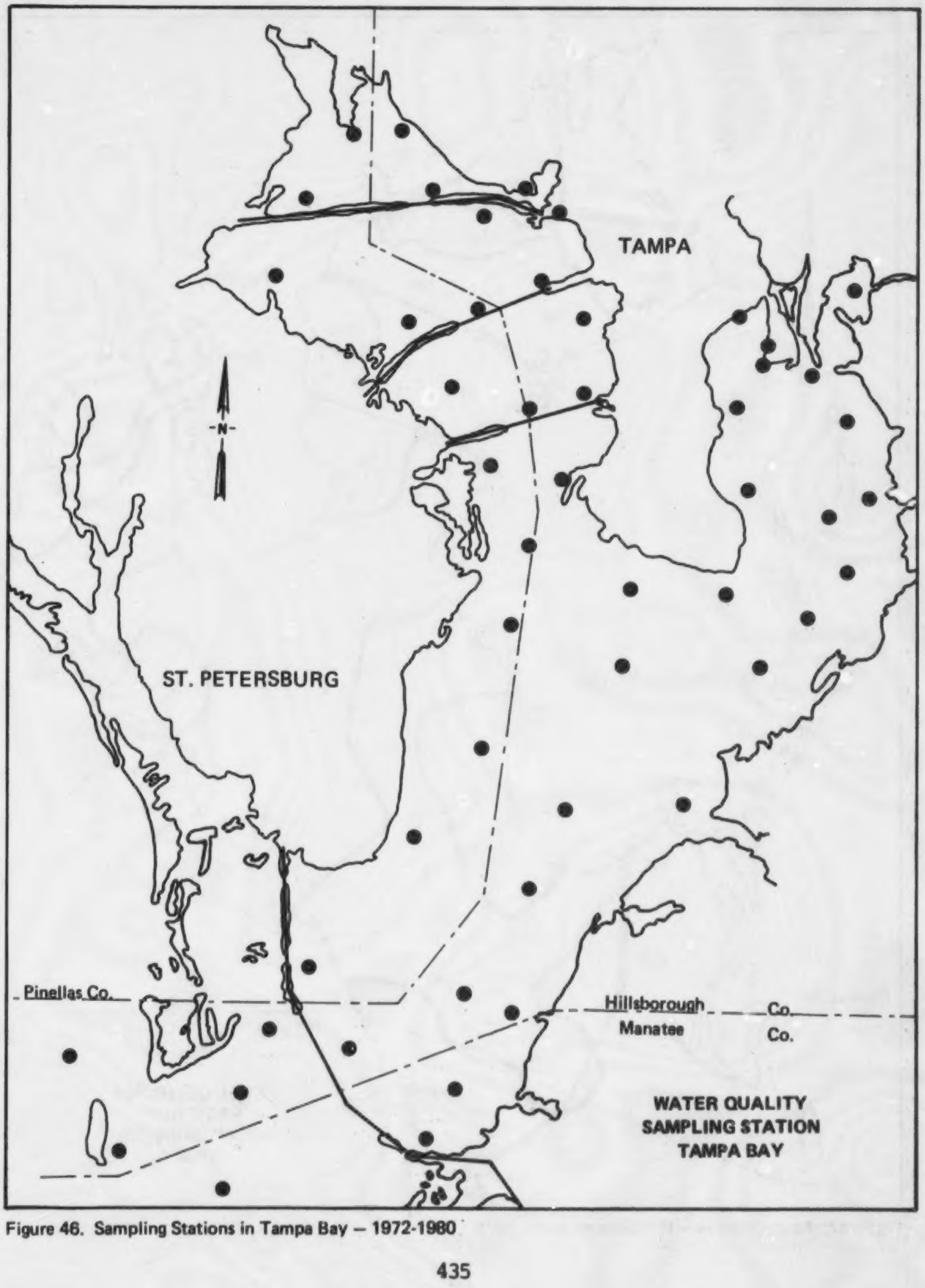
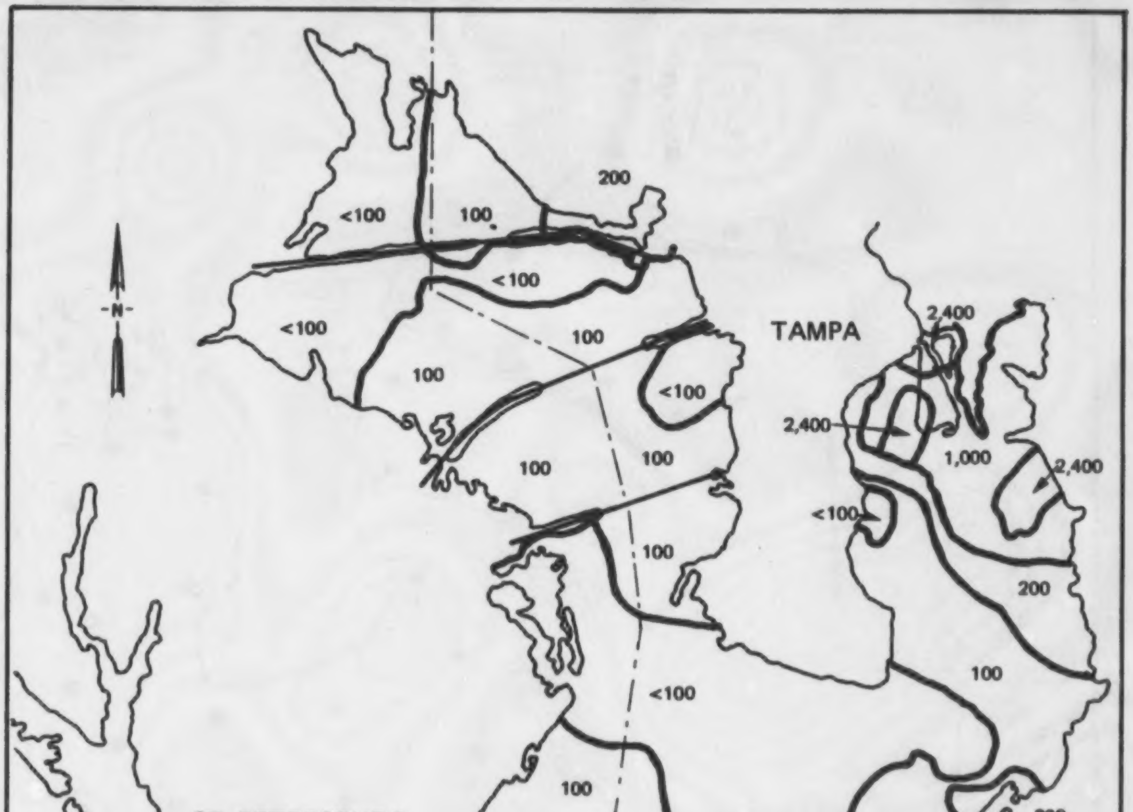

TAMPA
N
ST. PETERSBURG
Pinellas Co.
Hillsborough Co.
Manatee Co.
WATER QUALITY
SAMPLING STATION
TAMPA BAY

Figure 46. Sampling Stations in Tampa Bay — 1972-1980



200
<100
100
<100
<100
100
100
<100
TAMPA
2,400
2,400
1,000
2,400
100
100
<100
100
200
<100
100
ST. PETERSBURG
100
200
200
<100
100
Pinellas Co.
Hillsborough Co.
Manatee Co.
<100
FECAL COLIFORM
BACTERIA
(MF, CONFIRMED)
1978

Figure 47. Fecal Coliform – Hillsborough Bay – 1978

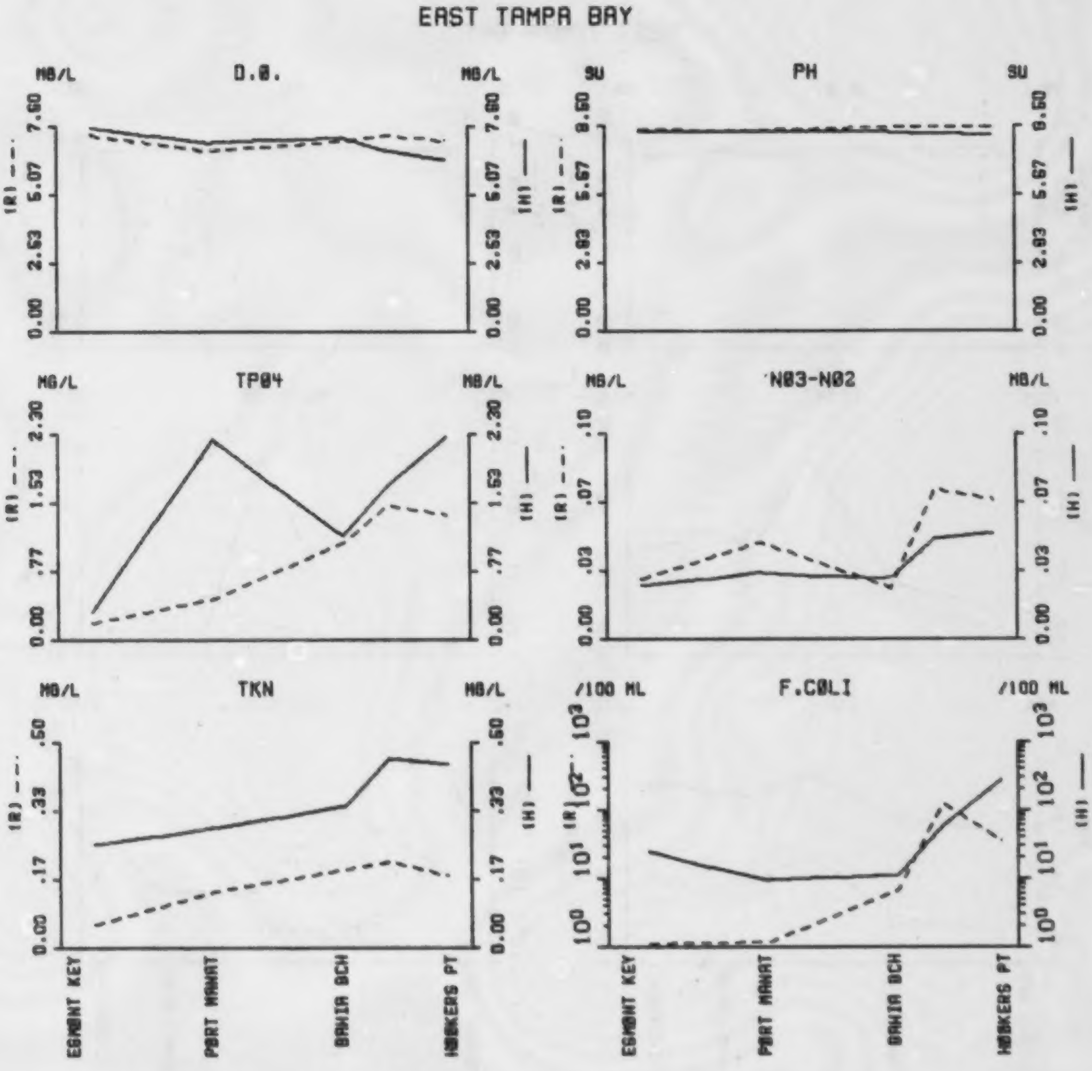


Figure 48. Water Quality Trends

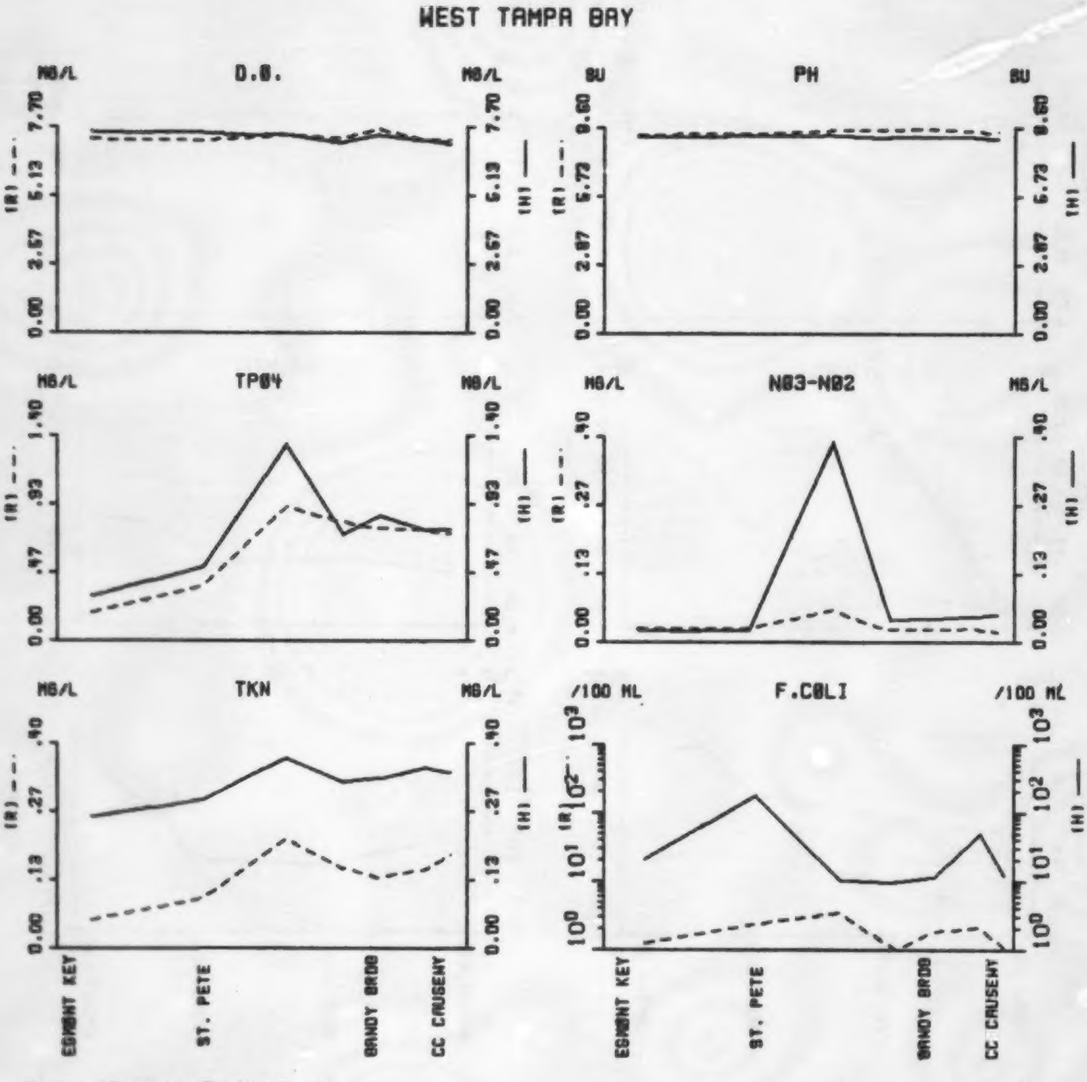


Figure 49. Water Quality Trends

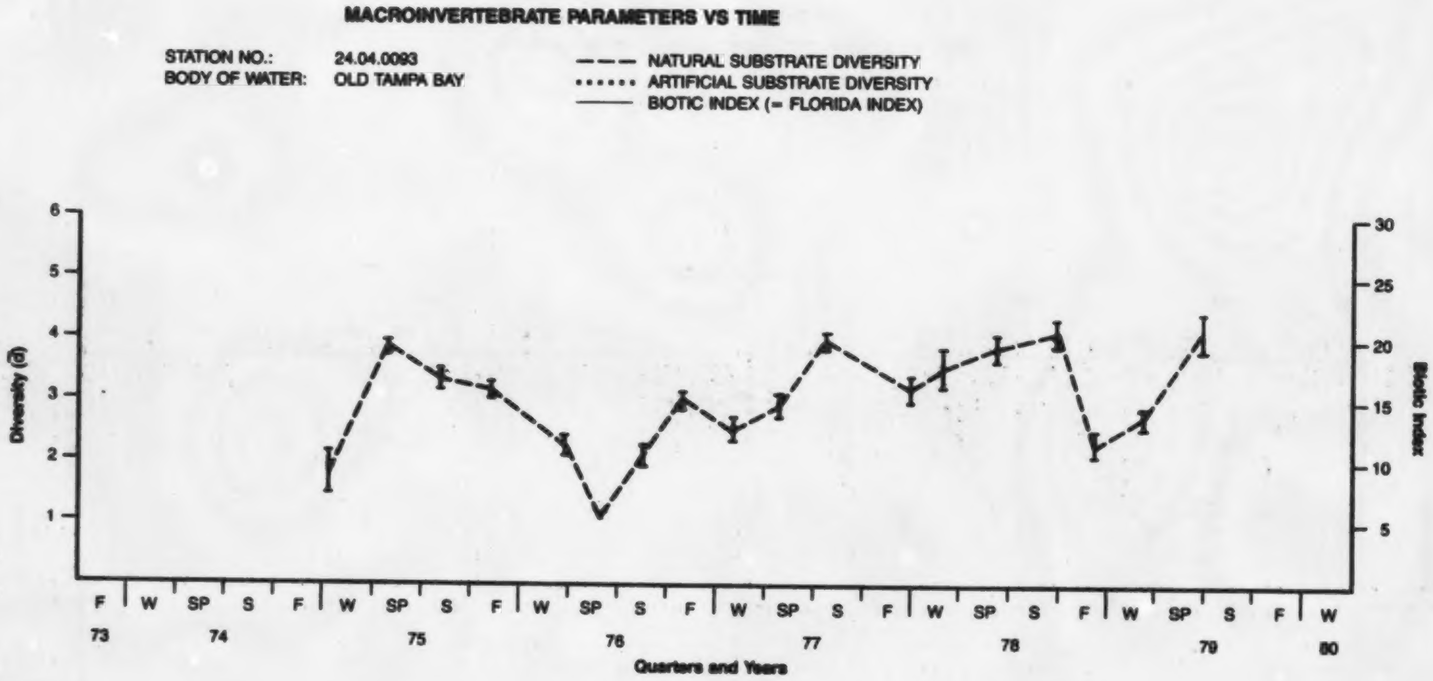


Figure 50. Macroinvertebrate parameters vs time.

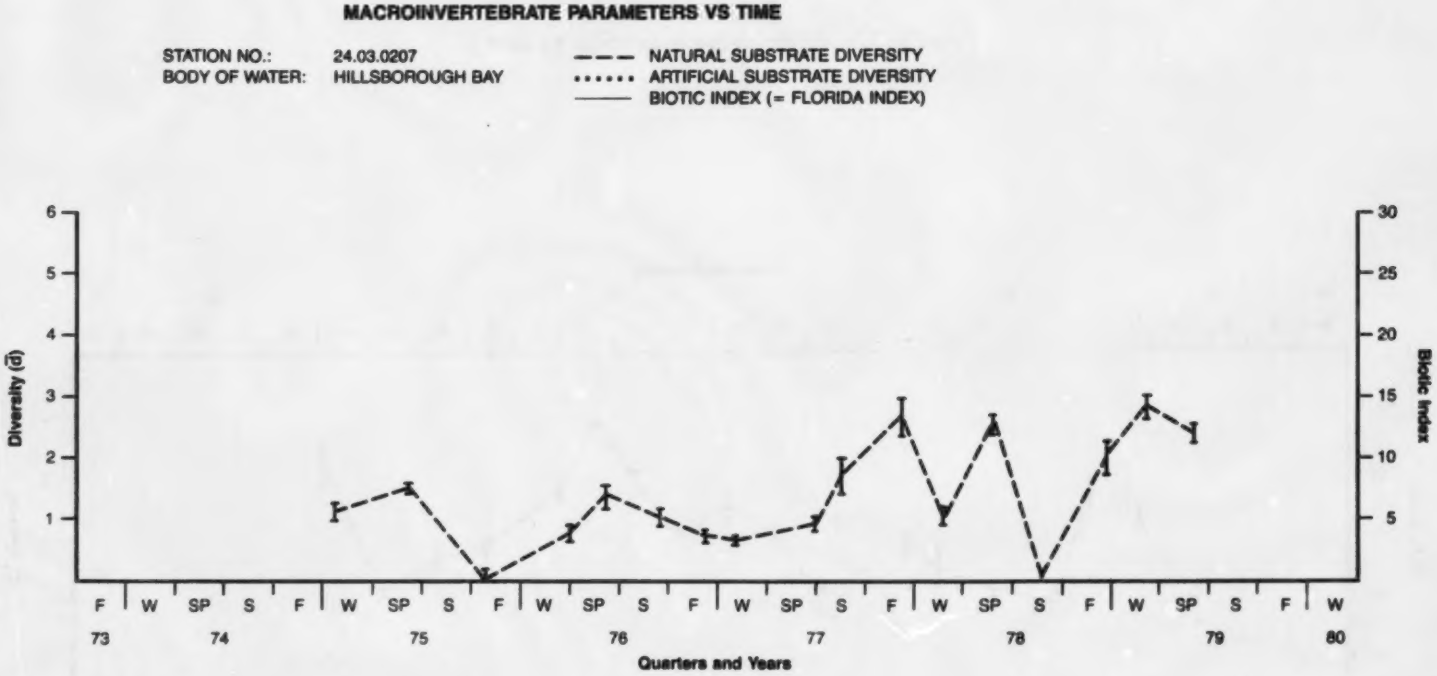


Figure 51. Macroinvertebrate parameters vs time.

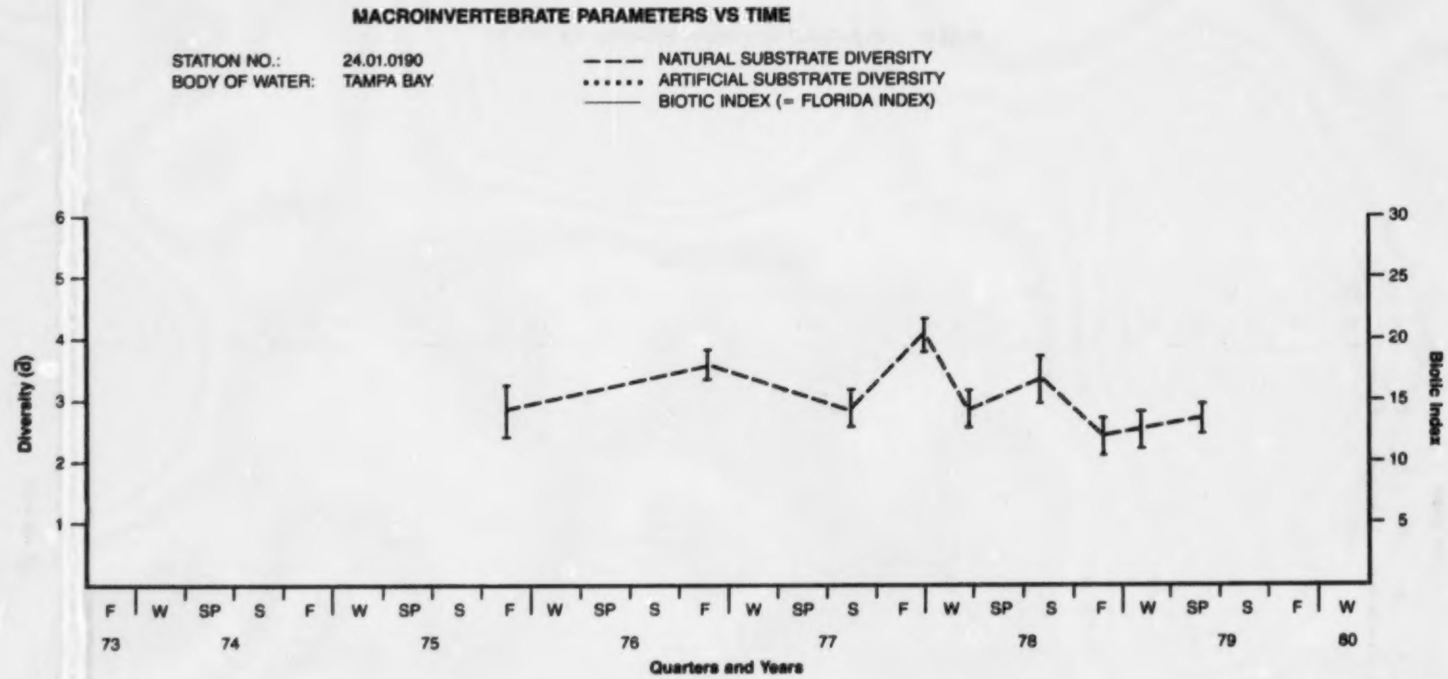


Figure 52. Macroinvertebrate parameters vs time.

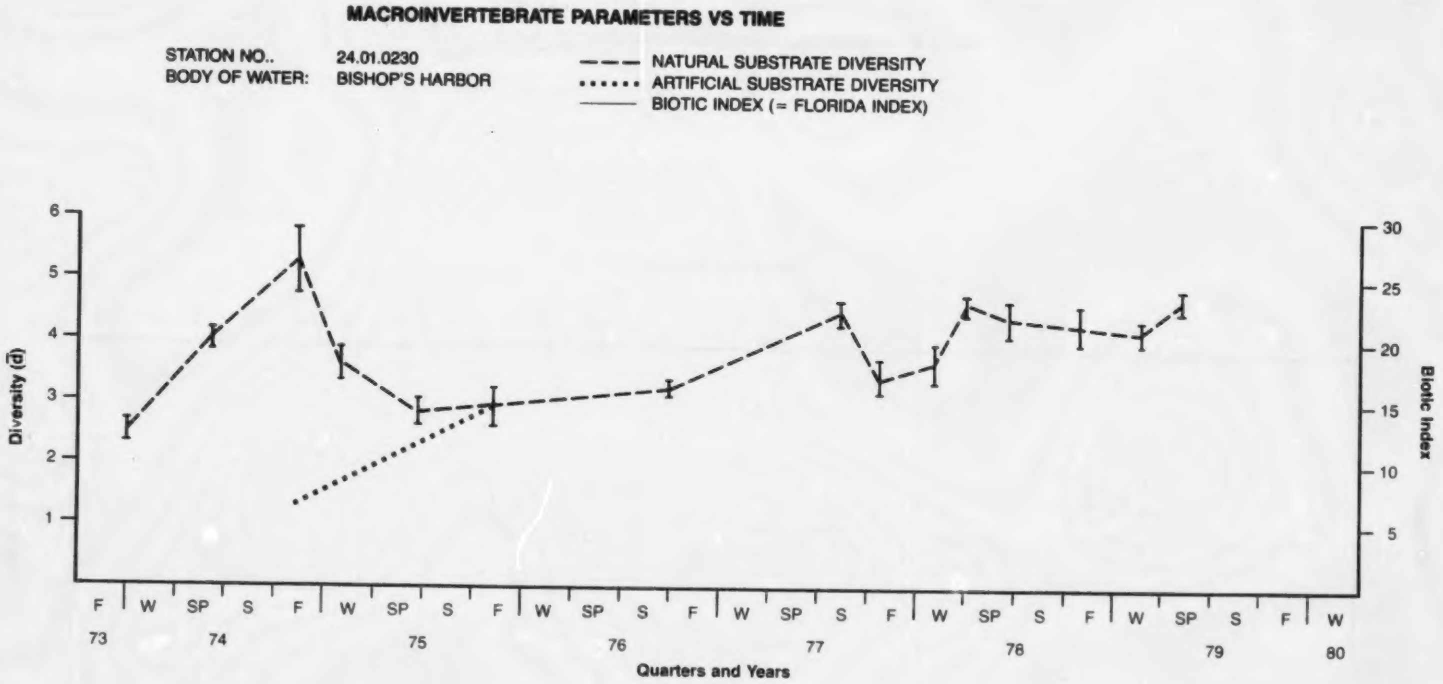


Figure 53. Macroinvertebrate parameters vs time.

biological study is presently underway to see what impact the AWT Plant is going to have on Hillsborough Bay. Unfortunately, several more months are required to gather and evaluate this date.

SEDIMENT

The sediments of Tampa Bay are laden with high levels of phosphorus (1-3 percent by weight) and nitrogen (.1-.5 percent by weight). These have been deposited in the bay system for many years. They have tended to be deposited along the ship channel and in the gyres caused by the currents in the Bay. These areas have heavy depositions, though sediment is present to some degree throughout the Bay. Without new depositions, it is reasonable to expect that these deposits could begin to stabilize. Unfortunately, the AWT Plant in Tampa was becoming operational and other municipal dischargers were upgrading or removing the discharges, a major ship channel-deepening project (Tampa Bay Ship Channel) was and still is underway. It is suspected that this project has today masked many of the beneficial effects of the upgrades on municipal sewage treatment works. This is believed to be caused by leaching of nutrients, especially nitrogen, from the muck in the sediments in the ship channel. Water quality data clearly show some increases in suspended solids in the last two to four years that coincide with the only significant dredging in the bay system (ship channel dredging). (This can be seen in Figures 54, 55, 56, 57, 58 for water quality in the Tampa Bay system.)

CONCLUSIONS

The effect of the special law has most probably been unduly expensive to citizens for the cost of sewage treatment in some areas covered by the law. There is most likely little justification for it based on water quality needs in those areas. In contrast, the law appears to have been beneficial to water quality in the upper reaches of Tampa Bay and the Pinellas County Peninsula. In all likelihood, water quality cause-effect relationships could not have been determined. It seems clear that an effort applied to demonstrating this cause-effect relationship did not do so in a fully satisfactory manner. The variables are clearly easier to predict in the Tampa Bay system than the Escambia Bay system and yet it is still a challenging task. Water quality has improved after advanced waste treatment at the City of Tampa Sewage Treatment Plant. Whether this high degree of treatment is enough is yet to be determined.

Another aspect of this law is reuse of water. The area covered by the law is a water-short area. While the law did not have reuse in mind when it was passed, it is having the effect of promoting reuse. This causes significant reductions in water use and thus, an important resource is not wasted. Strange as it may

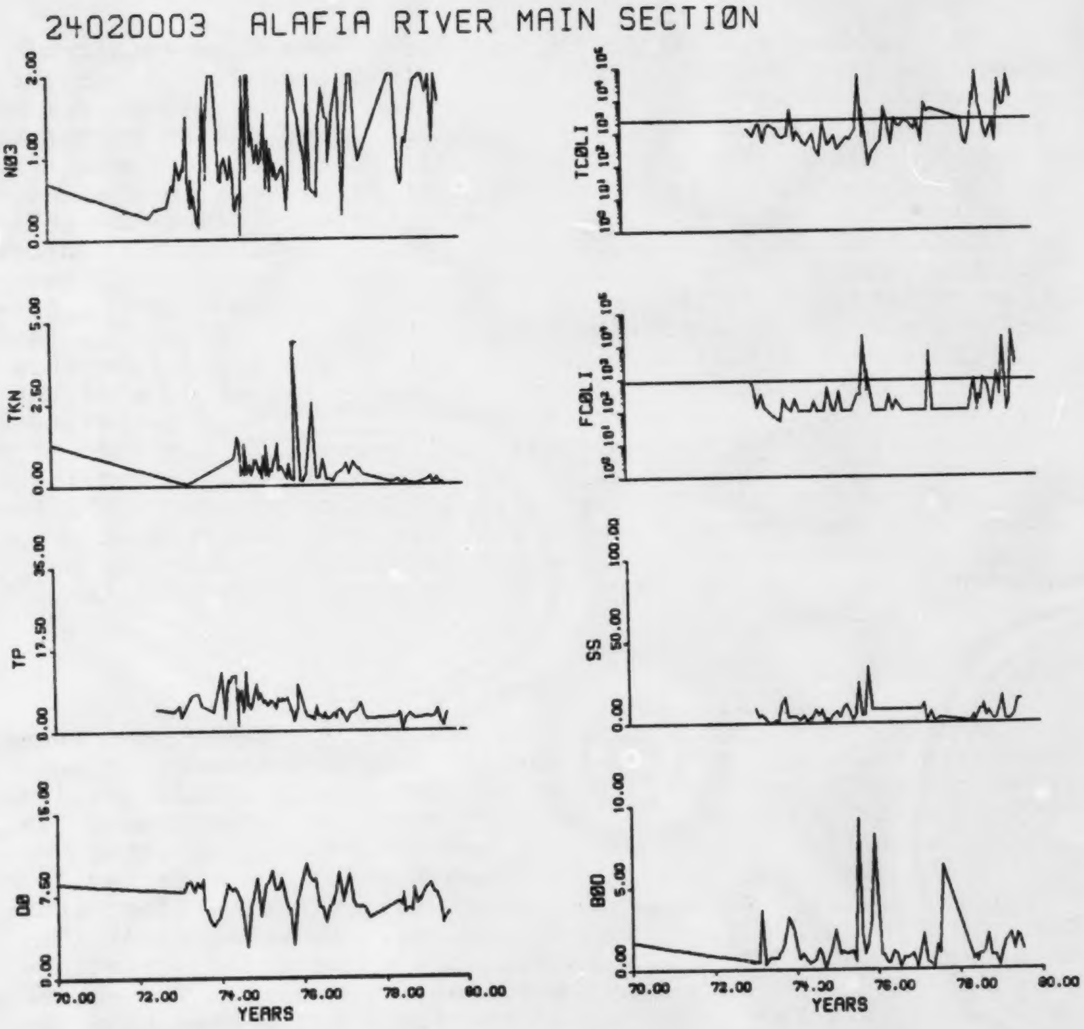


Figure 54. Water Quality — 8 Parameters — 1972-1979

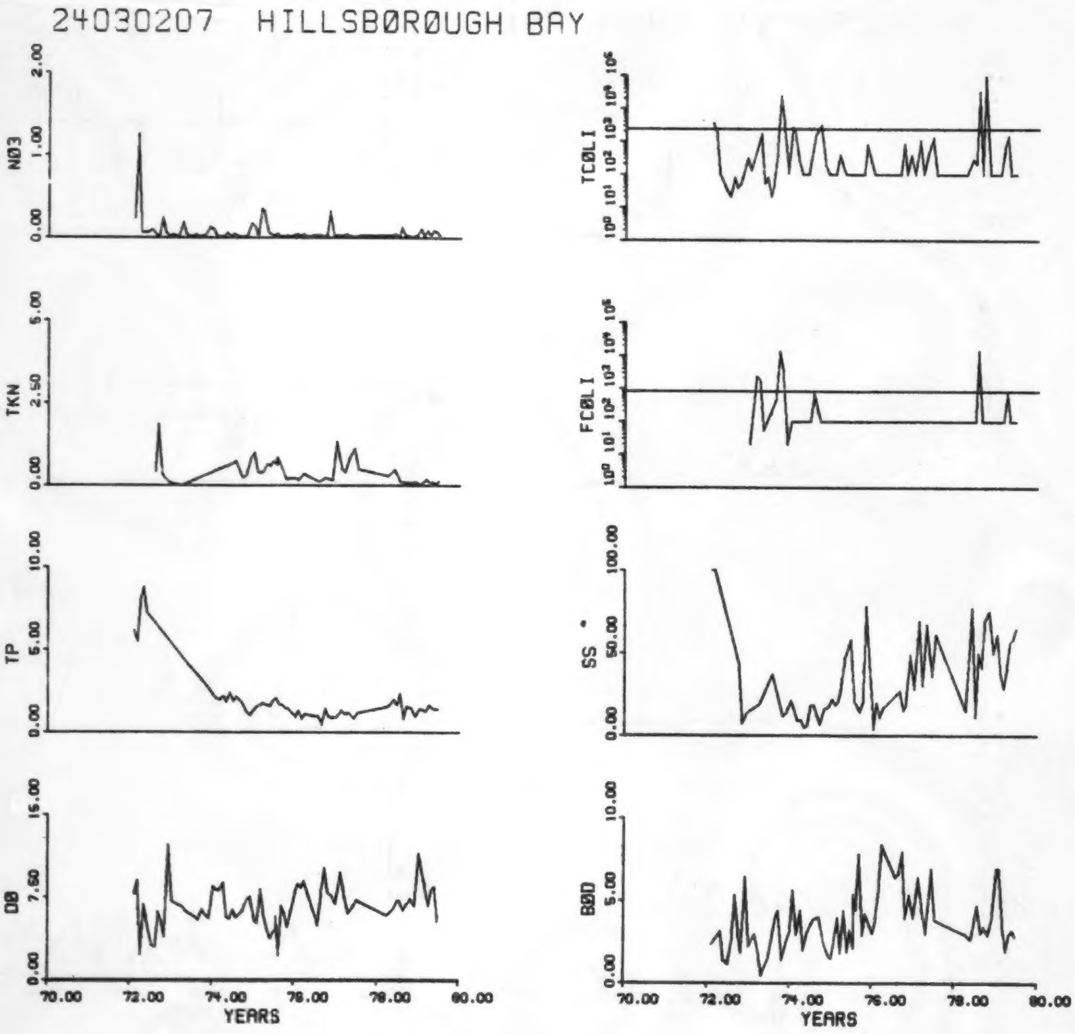


Figure 55. Water Quality – 8 Parameters – 1972-1979

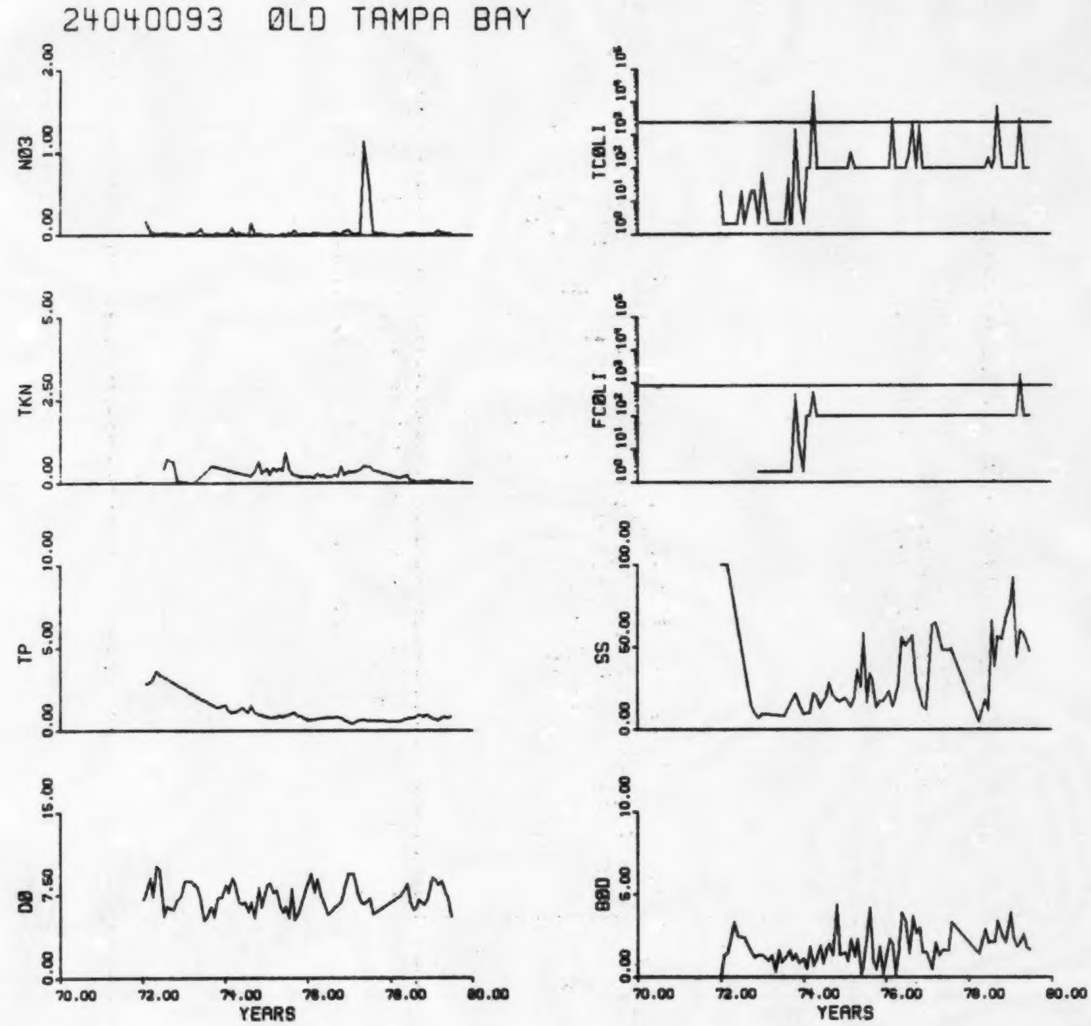


Figure 56. Water Quality — 8 Parameters — 1972-1979

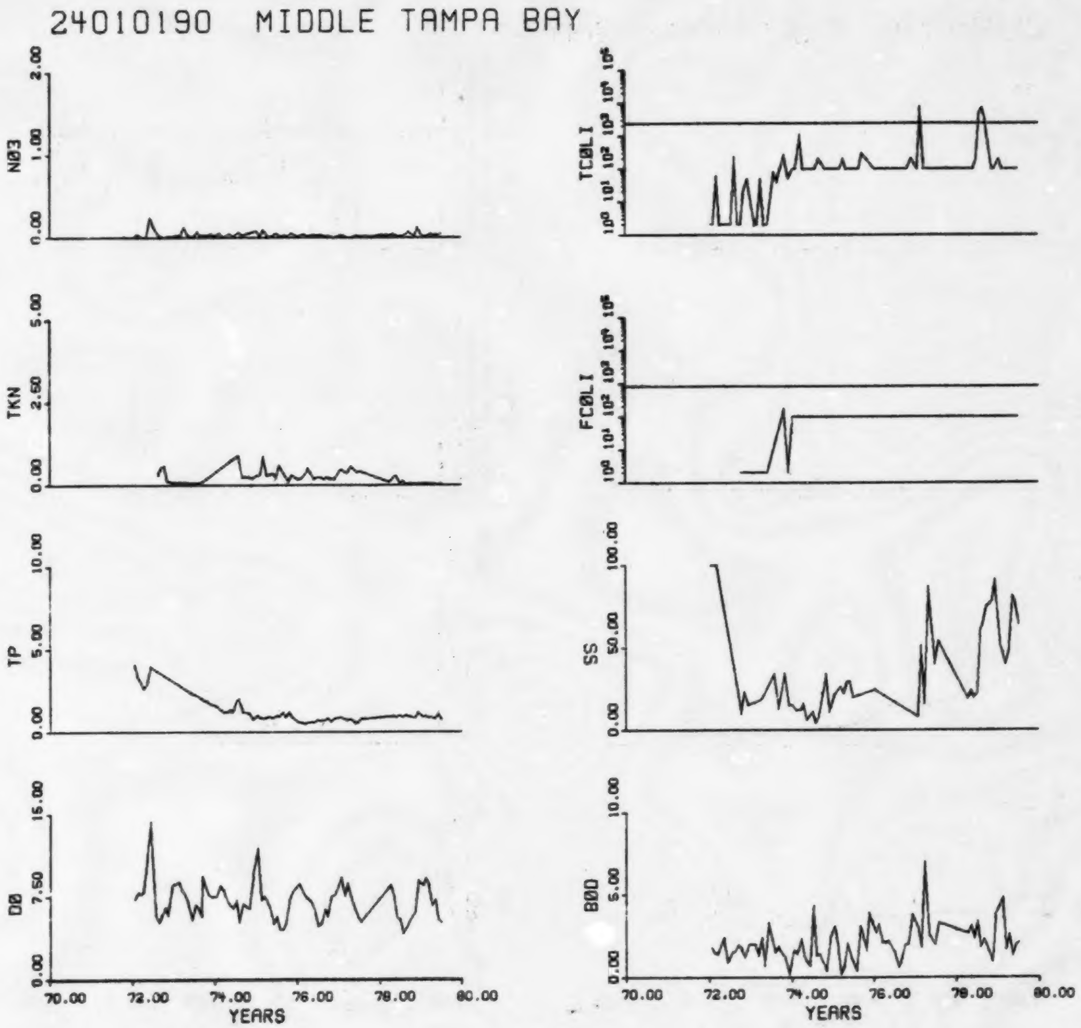


Figure 57. Water Quality — 8 Parameters — 1972-1979

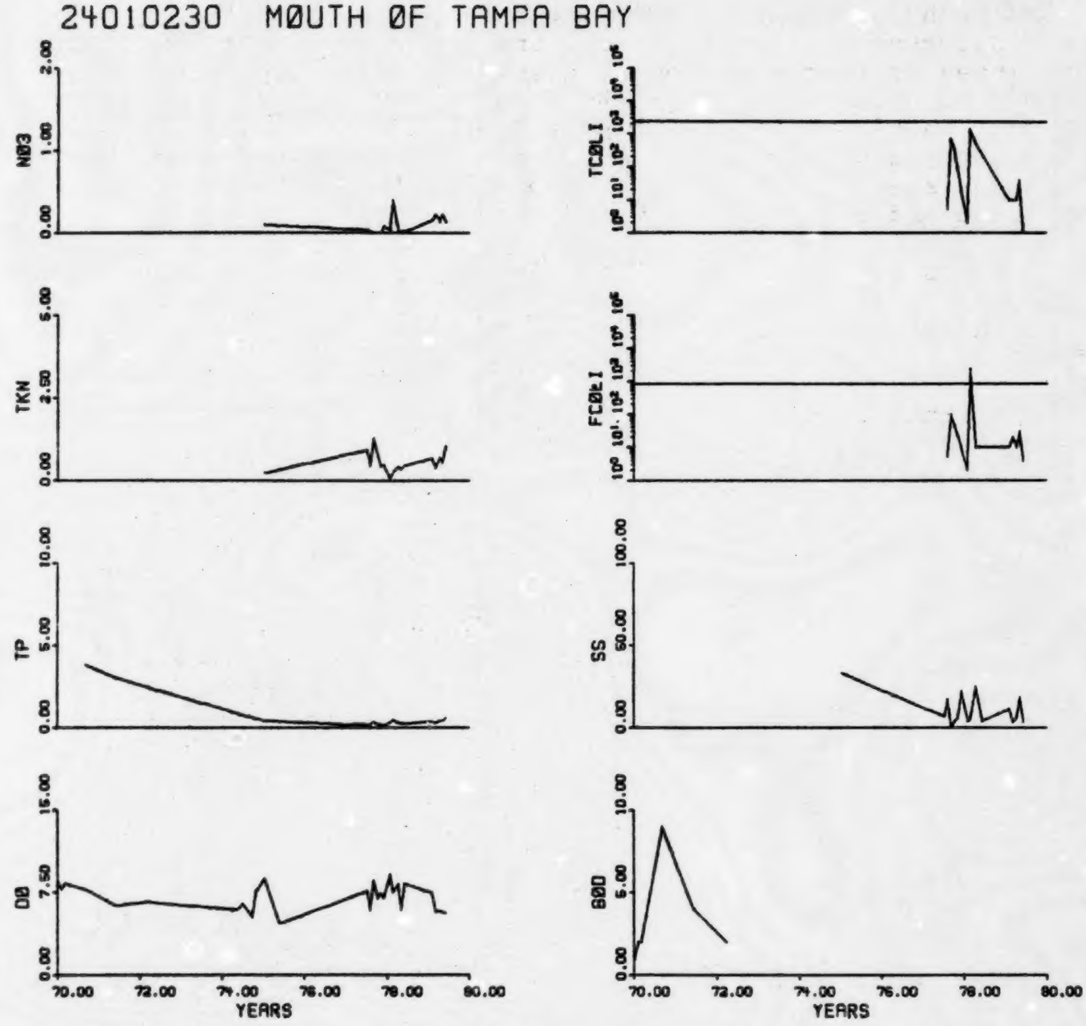


Figure 58. Water Quality – 8 Parameters – 1972-1979

seem, this law is causing implementation of a goal of the federal statute passed a year or so after the State law. The federal statute addresses the reuse of wastewater and this geographical area is reusing wastewater to save freshwater and avoid the cost of AWT.

If the nitrogen in the system does not become fixed after the ship channel dredging stops and the Tampa treatment plant continues to remove nitrogen, then, the three most probable causes of high nitrogen level in the water will be the Alafia River, leaching from sediments and urban stormwater runoff. The regulatory agencies will also have to reconsider the abatement strategy since a solution will most likely not become readily apparent and a manner of equity will become involved.

A massive study is being undertaken now by the City of Tampa funded in part by a National Urban Runoff Program Grant to determine the significance of urban stormwater and what can be done about it, if anything.

Further studies of the Alafia River to find the causes of relatively high nitrate levels will be done by FDER.

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INSTRUMENTATION AND AUTOMATION CONTROL
OF
MUNICIPAL SLUDGE TREATMENT FACILITIES

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INTRODUCTION

Dewatering and ultimate disposal of sludge resulting from primary and secondary treatment is a major problem in the United States. Costs range from 30% to 50% of the total treatment expense depending on the plant size and the technology applied. One potential method of reducing costs in this area is the application of instrumentation and automation to process control of dewatering and ultimate disposal. Unfortunately only minimal application of instrumentation and automation has been made in this area. Thus the U.S. EPA entered into a joint program with the Minneapolis-St. Paul Metropolitan Waste Control Commission (MWCC) to investigate, develop, and demonstrate process control and automation of dewatering and incineration of domestic sewage sludge.

Based on frequency of use in the U.S. and availablility of full-scale parallel lines of equipment at MWCC, the decision reached was to concentrate on the following four processes: gravity thickening, chemical conditioning, vacuum filtration, and multiple-hearth incineration. The data on automation of a gravity thickener was presented at the last U.S.-Japan Conference in October 1978 (1). The full details of the study are in an EPA report (2). This paper will deal with the studies conducted on the other three unit operations.

The studies here involve determination based on literature sources, laboratory tests and field tests of control strategies for each unit operation.

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including an evaluation of the instrumentation and automation necessary to implement the control strategy. As the studies were conducted on full-scale equipment side by side with identical manually controlled equipment the cost effectiveness of the automation could be determined. Additionally the three unit operations are connected in series and were evaluated as an overall treatment line rather than individual units. That is, a control strategy was developed for economic optimization of the overall sludge handling line rather than optimization of each unit separately.

Unfortunately all of the studies have not been completed at this time. Therefore this paper will deal with the progress made to date on each phase of the project.

FACILITY DESCRIPTION

A. General

The Metropolitan Waste Control Commission (MWCC), which was formed in 1970, currently operates 16 wastewater treatment plants in the Minneapolis-St. Paul metropolitan area. Average flows at the facilities fall in the range of approximately 0.2 to 200 mgd. All of the work described in the following sections was conducted at the Seneca Wastewater Treatment Plant which is located on the Minnesota River in the City of Eagan, Minnesota. This facility serves a number of suburban communities south of Minneapolis.

The Seneca Facility was placed in operation in July 1972 with an average design capacity of 24 mgd. During 1979 the flow averaged approximately 13.5 mgd. Based on MWCC records approximately 5% of the influent flow is from industrial sources. The treatment scheme consists of screening, grit removal, primary sedimentation, complete mix activated sludge, final sedimentation, and chlorination prior to discharge to the receiving stream.

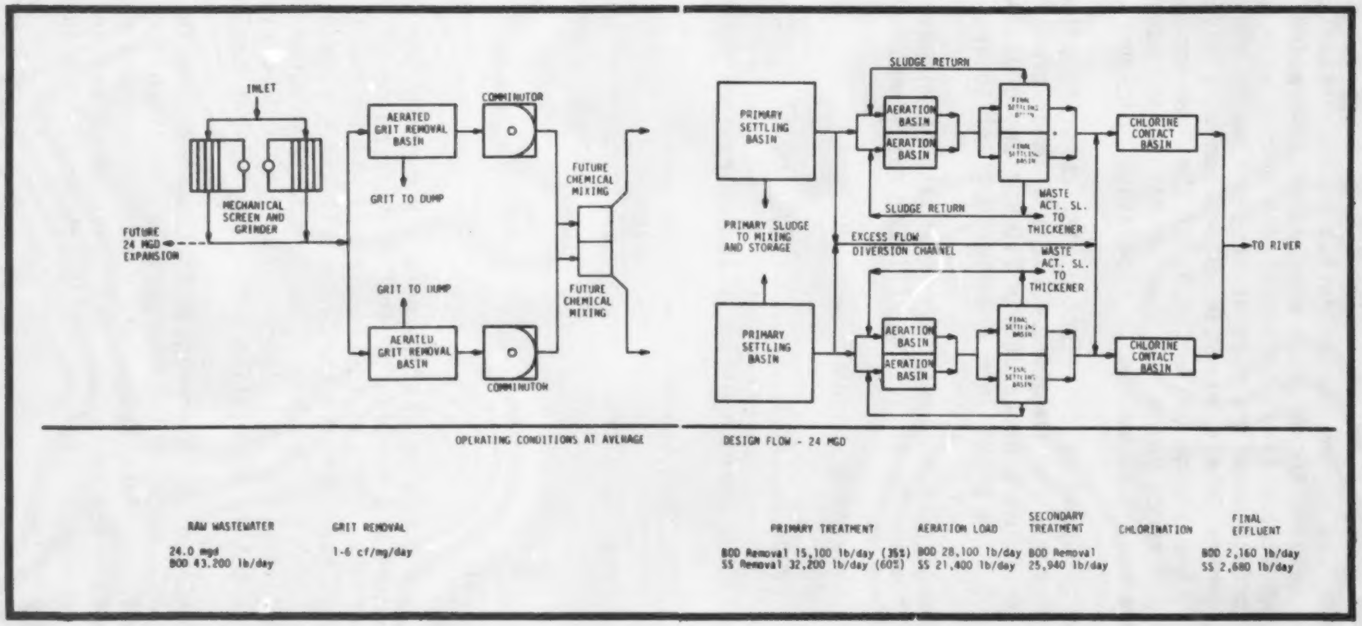
B. Liquid Train

The liquid treatment portion of the plant is divided into two parallel treatment trains each with a nominal capacity of 12 mgd. The following paragraphs describe one of the parallel treatment trains; however, considerable flexibility does exist and the flows can cross over between trains at several points as illustrated in Figure 1.

The raw wastewater is screened to remove large debris which might interfere with mechanical equipment downstream. Grit is removed in an aerated grit chamber with a detention time of approximately 15 minutes.

Subsequent to grit removal the wastewater flows by gravity to the primary sedimentation basin. This basin is square and has a side wall depth of 12 feet. The surface overflow rate at design flow is 512 gpd/sq. ft. The basin is equipped with a center inlet and sludge plows for central sludge withdrawal. Surface skimming equipment is provided for removal of scum.

FLOW DIAGRAM-LIQUIDS PROCESSING


INLET
MECHANICAL SCREEN AND GRINDER
FUTURE 24 MGD EXPANSION
AERATED GRIT REMOVAL BASIN
GRIT TO DUMP
GRIT TO DUMP
AERATED GRIT REMOVAL BASIN
COMMINUTOR
COMMINUTOR
FUTURE CHEMICAL MIXING
FUTURE CHEMICAL MIXING
PRIMARY SETTLING BASIN
PRIMARY SLUDGE TO MIXING AND STORAGE
PRIMARY SETTLING BASIN
SLUDGE RETURN
AERATION BASIN
AERATION BASIN
FINAL SETTLING BASIN
FINAL SETTLING BASIN
SLUDGE RETURN
WASTE ACT. SL. TO THICKENER
EXCESS FLOW DIVERSION CHANNEL
WASTE ACT. SL. TO THICKENER
AERATION BASIN
AERATION BASIN
FINAL SETTLING BASIN
FINAL SETTLING BASIN
CHLORINE CONTACT BASIN
CHLORINE CONTACT BASIN
TO RIVER
OPERATING CONDITIONS AT AVERAGE
DESIGN FLOW - 24 MGD

RAW WASTEWATER	GRIT REMOVAL	PRIMARY TREATMENT	AERATION LOAD	SECONDARY TREATMENT	CHLORINATION	FINAL EFFLUENT
24.0 mgd BOD 43.200 lb/day	1-6 cf/mg/day	BOD Removal 15,100 lb/day (35%) SS Removal 32,200 lb/day (60%)	BOD 28,100 lb/day SS 21,400 lb/day	BOD Removal 25,940 lb/day		BOD 2,160 lb/day SS 2,680 lb/day

FIGURE 1. SENACA TREATMENT PLANT

The primary effluent is distributed by gravity to two mechanically aerated basins each with a volume of approximately 730,000 gal. Total aeration time at design flow, excluding sludge recycle, is approximately 3 hours. The dissolved oxygen concentration in the basin is adjusted by controlling the aerator submergence with adjustable weirs. The mixed liquor discharges to two square final sedimentation basins with side wall depths of 15 feet. The surface overflow rate at design flow is 490 gpd/sq. ft. These basins utilize center inlets and skimming equipment similar to the primary basins; however, the sludge collector is of the "vacuum cleaner" type. Both variable and fixed speed pumps are used to return and waste sludge as required. Excess sludge is wasted to floatation thickeners.

The plant effluent is disinfected with chlorine prior to discharge to the receiving stream. The chlorine contact chamber provides a nominal retention time of 22 minutes at design flow prior to discharge through approximately 0.5 mile of 84 inch diameter outfall sewer.

The discharge permit granted by the state of Minnesota requires that the Seneca plant meet the following effluent requirements:

Suspended Solids	-	30 mg/l
BOD	-	25 mg/l
Fecal Coliform	-	200/100 ml

During 1979 the average effluent concentrations were:

Suspended Solids	-	20
BOD	-	16
Fecal Coliform	-	17 (average of monthly geometric means)

C. Sludge Train

1. General

The solids processing operations are housed in the Seneca solids processing building and include floatation thickening of waste activated sludge, vacuum filtration and incineration. All of the primary and waste activated sludges generated in the Seneca liquid treatment trains are processed along with sludges hauled to Seneca from other MWCC facilities. During 1979 approximately 38 million gallons of liquid sludge was processed including approximatley 10 million gallons which was hauled to the plant.

The schematic diagram presented in Figure 2 describes the sludge dewatering trains. Each of the two parallel treatment trains consist of chemical conditioning facilities, one vacuum filter and one incinerator. The two trains can be cross connected just upstream of the incinerators. The ash from each incinerator is slurried and transported to a lagoon for gravity dewatering and storage.

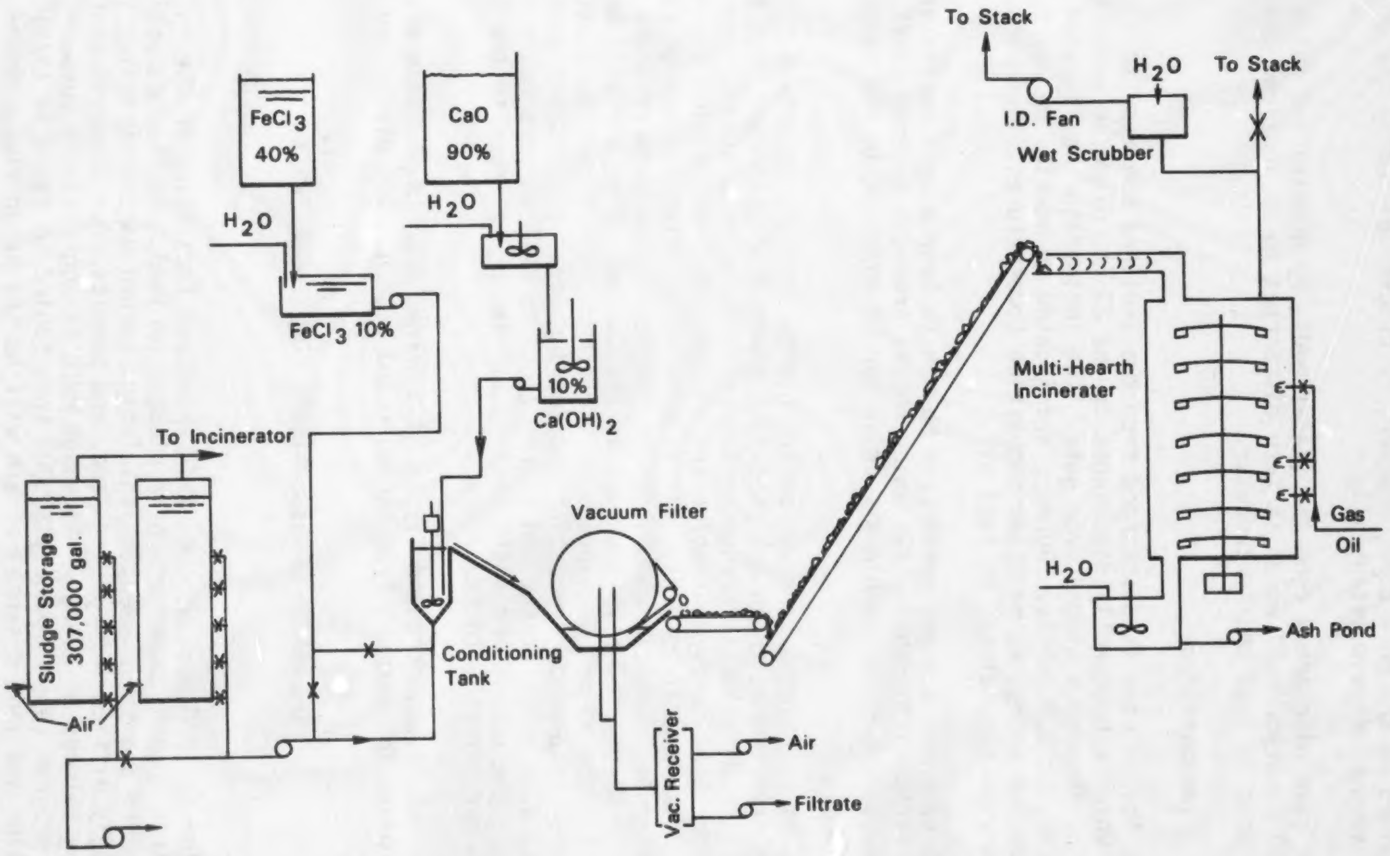

To Stack
H2O
To Stack
I.D. Fan
Wet Scrubber
FeCl3
40%
CaO
90%
H2O
H2O
FeCl3 10%
10%
Ca(OH)2
Multi-Hearth
Incinerater
To Incinerator
Vacuum Filter
Gas
Oil
Sludge Storage
307,000 gal.
H2O
Ash Pond
Conditioning
Tank
Air
Vac. Receiver
Air
Filtrate

FIGURE 2. SLUDGE PROCESSING AND ULTIMATE DISPOSAL FLOW SHEET

The vacuum system is common to both the form and drying zones of the filter. Under normal operating conditions a vacuum of 20 to 25 inches Hg can be maintained in the form zone; however, if cracks develop in the drying zone, the form vacuum drops substantially.

The cake discharges from the filter media by gravity and falls onto a horizontal conveyor belt which in turn discharges to an inclined conveyor which is used to load the incinerator.

5. Incineration

The filter cake is discharged from the inclined conveyor to a screw conveyor which subsequently discharges to the 23 ft. diameter seven hearth incinerator through a hinged drop gate. The incinerator has a rated capacity of 7 ton/hr. at 22% solids. Burners are located on hearths no. 3, 5, and 6 to provide the energy to maintain combustion temperatures. The burners can be fueled with natural gas or fuel oil.

The cake and ash are moved from hearth to hearth by a series of rakes or plows attached to rabble arms which rotate around a central shaft. The rotational speed of the rabble arm drive can be adjusted in the range of 0 to 2 RPM.

Ash is discharged from the bottom hearth into a slurry tank. The ash slurry is then pumped to an ash pond for intermediate storage. Ash balls which consist of volatile sludge solids covered and insulated by char are removed on a bar screen just above the slurry tank. Combustion air is supplied from several sources. Air is metered into the furnace at each burner to ensure complete combustion of the fuel. The induced draft fan maintains a slight negative pressure in the furnace; thus air will be pulled into the furnace at all openings. An atmospheric air damper is located on hearth seven to provide combustion and cooling air. Significant quantities of air may also enter at the ash slurry chute as well as port holes that are left open. The rabble arm shaft cooling air can be returned to the furnace or discharged directly to the stack.

The hot gases, soot and fly ash discharged from the furnace are ducted at a wet plate impingement type scrubber and subsequently discharged to the stack.

OPERATING CONSIDERATIONS FOR SLUDGE TRAIN

A. General

Prior to 1970 most of the small treatment facilities in the metropolitan area disposed of their sludges on land. As the metropolitan area expanded and environmental legislation turned more restrictive it became increasingly difficult to obtain land, and permits, for disposal of the increasing volumes of sludge. Although MWCC is aggressively pursuing a land disposal program (approximately 20,000 tons hauled to farms in 1979) incineration and land disposal of ash will be the major sludge disposal scheme in the forseeable future.

2. Storage

The primary sludge and air floatation thickened waste activated sludge along with the hauled sludges are discharged to two holding tanks. The tanks are 40 feet deep and have a combined capacity of approximately 615,000 gal. The contents of the tanks may be mixed by introducing air near the bottom on a timed or manual basis. The mixing air is vented to the incinerators to prevent odor problems.

Sludge can be withdrawn from the tanks at seven points along the side wall between 0 and 35 feet above the tank floor. In addition supernatant removal is provided with the supernatant returned to the primary influent. The sludge is removed from the storage tanks using positive displacement pumps of the progressing cavity type and transported to the conditioning tank through approximately 150 feet of 6 inch dia. pipe.

3. Chemical Conditioning

Ferric chloride and lime are used to condition the sludge prior to dewatering. Ferric chloride is delivered to the facility in solution form at a nominal concentration of 40% $FeCl_3$. The concentrated solution is diluted to approximately 10% $FeCl_3$ in a 2000 gal day tank. Air operated diaphram pumps are used to transport the diluted $FeCl_3$ solution to the point of addition to the sludge. Prior to the initiation of this study the $FeCl_3$ was added to the sludge at the base of the conditioning tank. The point of addition was changed to allow for some additional mixing prior to lime addition. The current addition point is approximately 25 feet downstream of the sludge pump and approximately 125 feet upstream of the conditioning tank.

Lime is delivered to Seneca by railcar and stored in a large silo. The delivered material is approximately 90% CaO by weight. The CaO is converted to $Ca(OH)_2$ using standard slaking apparatus. The $Ca(OH)_2$ slurry is maintained at approximately 10% CaO by weight and is transported to day tanks by gravity. Positive displacement pumps, of the progressive cavity type, are used to transport the lime slurry to the conditioning tank.

The conditioning tank has a volume of approximately 200 gal. and is equipped with a variable speed mixer. The mixing impeller consists of 3 vertical, triangular shaped blades and rotates in the range of 25 to 84 RPM. The conditioned sludge discharges from the tank over a weir approximately 16 in. long and flows down an inclined pan to the filter vat.

4. Vacuum Filtration

A conventional rotary drum filter, manufactured by Dorr-Oliver, is used to dewater the conditioned sludge. The drum is 12 ft. in diameter and has a total area of 528 sq. ft. The drum is covered with polyethylene media and can be rotated in the range of 9 to 30 RPH.

The filter vat is equipped with a variable speed, paddle type agitator which can be operated in the range of 5.7 to 13 cycles per min.

The Seneca liquid trains are currently operated at approximately 50% of design capacity; however, because sludge is hauled from several other treatment facilities, the sludge processing trains are operated at their practical capacity. In these circumstances the goal for the sludge processing trains is "process the maximum quantity of sludge".

B. Sludge Processing Limitations

1. Chemical Conditioning

Each of the two sludge processing trains at Seneca consists of three processes in series - chemical conditioning, vacuum filtration and incineration. Depending on circumstances, any one of the three processes can be the rate limiting step.

Prior to the initiation of this project the filter operator had no quantitative criteria on which to judge the effectiveness of chemical conditioning. If the cake discharged from the filter cloth and a reasonable cake production rate was maintained the sludge was judged to be adequately conditioned. If problems were encountered, cake discharge or production rate, changes were made in the ferric chloride and/or lime dose. Each operator's response was based largely on his previous experience. In terms of meeting the plants performance goal the natural tendency was to overdose.

Because sludges from several plants were/are being processed at Seneca, problems were encountered even when the chemical rates were extremely high. As the physical and chemical characteristics of the raw sludge change the dewatering characteristics of the conditioned sludge may change significantly even when overdosed.

Rapid changes prevent both the filtration and incineration processes from maintaining steady state conditions. The operators have observed that old sludges are much more difficult to condition than fresh sludges; however, if an inventory in the sludge holding tank is maintained at a minimum, the impact of batch discharges from other plants on sludge properties is increased.

Because of the nature of the material used for conditioning, the chemical feed pumps fail periodically. The impurities in the lime accelerate the wear of both the stator and rotor of the progressive cavity pumps; and, impurities in the ferric chloride prevent the check valve from sealing properly allowing back flow. When the sludge is overdosed significantly it is difficult to recognize the gradual deterioration of pump performance and acute failures are more likely to occur. When one of the pumps fails, the operator most likely will not become aware of this fact until the cake does not discharge from the cloth and pump speed adjustments do not alleviate the problem. Such failures require a full shut down of the filter and incinerator until another pump is put into service, the filter vat and conditioning tank are purged of the poorly conditioned sludge, and the filter cloth is cleaned.

As the subsequent chapters will point out in detail many of the problems related to chemical conditioning can be ameliorated by monitoring the physical and chemical characteristics of the raw and conditioned sludges.

2. Vacuum Filtration

Prior to the initiation of this study the vat level was not automatically controlled on either of the filters. The operator thus had four options to change the cake yield: (1) change drum speed, (2) change vacuum, (3) change sludge pumping rate and (4) change chemical conditioning. As a practical matter the vacuum is held at maximum value at all times. If the sludge is overdosed moderate changes in chemical feed rates do not affect a significant change in dewatering characteristics. The four options were thus reduced to changes in the drum speed and/or pumping rate. Change in drum speed generally required a change in sludge pumping rate. After several revolutions the vat level reaches a new equilibrium. Small step changes are required to ensure that the vat does not overflow or go so low as to expose the form zone on the filter. Because of the lack of simple automated control and the uncertain nature of the manual control scheme available,it is unlikely the operating goal was attained a significant portion of the time.

Because of the variable nature of the raw sludge, the conditioning process and filter operation itself, it is extremely difficult for an operator to maintain a steady output of cake with uniform burning characteristics. The data presented in Figure 3 were collected during a 24 hour period in 1975. The solids content of the cake varied from 14% to 25.5% and the volatile solids content fell in the approximate range of 53% to 68%. For a six hour period the sludge was returned to the plant headworks because a cake which would discharge from the filter cloth could not be formed. During a similar sampling period in 1976 the solids content varied in the range of approximately 20% to 28% and the filter yield varied by a factor of three.

Experience with the sludges processed at Seneca indicates that the pH of the conditioned sludge must be raised to approximately 12.0 to obtain adequate dewatering characteristics. At this high pH there is a definite tendency to deposit lime on all surfaces including the filter cloth and vacuum system. When the sludge is overdosed it is reasonable to believe that this plating tendency increases thus causing increased downtime for acid cleaning.

3. Incineration

Because there is no cake storage capacity at Seneca, the incinerator in each train must burn the cake as produced. This must be accomplished within the constraints of meeting air pollution standards and maintaining hearth temperatures below 2000°F to protect the furnace structure. In addition, it is required that good volatile destruction be obtained and the formation of ash balls be minimized.

Because of the variable nature of the filter output it is extremely difficult to maintain steady burning conditions in the incinerator. The situation is further complicated by the fact that the analog temperature control system, designed to control the burners, does not function properly because of interaction between the hearths. In addition, the excess oxygen

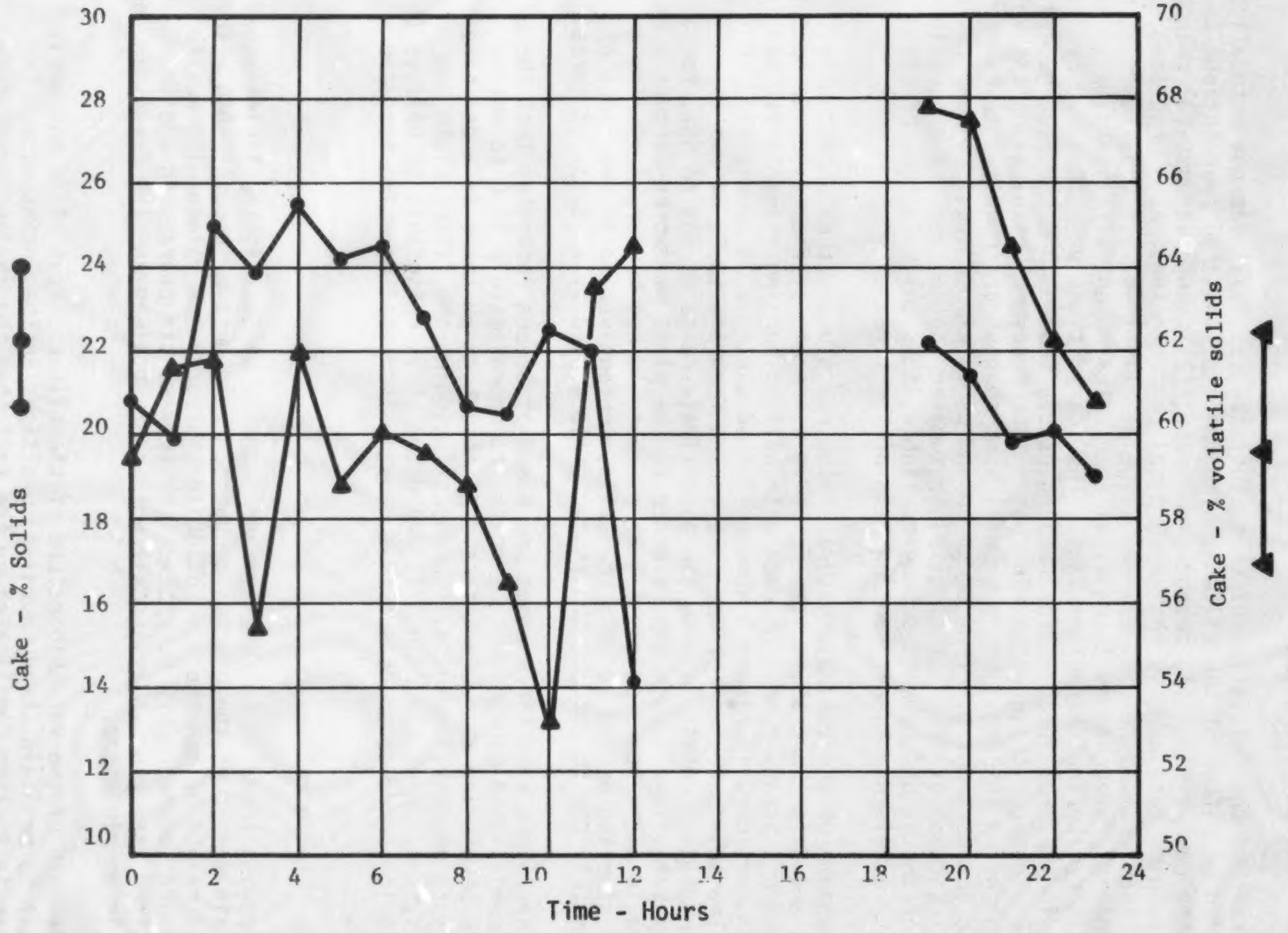


FIGURE 3. VARIABILITY OF CAKE DISCHARGE FROM VACUUM FILTER UNDER MANUAL CONTROL

level is not measured and therefore cannot be easily controlled.

In the existing system the operators have few tools available to them to attain the operating goal and must make judgements based on past experience and process understanding. It is thus not surprising to find that the performance of the entire sludge processing train can and does vary from shift to shift.

C. Cost

The total operating cost for sludge dewatering and incineration at Seneca is the sum of the costs for chemicals, energy, and labor. In the following paragraphs an attempt is made to characterize portions of these cost categories for the Seneca solids processing trains for the year 1979. All costs are in terms of January 1980 dollars.

Based on Seneca operating records for 1979 approximately 38 million gallons of sludge with an average solids content of 5% was dewatered to yield approximatley 88 million pounds of cake with an average solids content of 21%. The quantities and costs for conditioning chemicals were as follows:

Matl.	lb.	Avg. Dose-%	1979 Cost Total-$	1979 Cost $/Ton Sludge-Solids
$FeCl_3$	1,362,000	8.5	95,340	11.95
CaO	3,358,850	21	96,660	12.11
Total	4,720,850	29.5	192,000	24.06

Natural gas is supplied to the Seneca facility on an interruptable basis. As the system wide demand for gas increases during the cold weather months the gas supply is terminated and an alternate fuel (oil) must be used in the incinerators. During 1979 incinerator fuel quantities and costs were as follows:

Fuel	Quantity	1979 Cost Total-$	1979 Cost $/Ton Sludge-Solids
Natural Gas	117,188 mcf	140,625	17.62
Oil	149,020 gal	119,214	14.94
Total	-	259,839	32.56

A total of five positions are involved in the operation of the solids building as follows:

Position	% Time	Annual Cost (all shifts and fringe benefits)
Building Operator (foreman)	90	$93,000
Filter Operator	100	$102,000
Incinerator Operator	100	$102,000
Laborer	100	$91,000
Laborer	50	$45,000
Total	-	$433,000

Based on 1979 records the labor cost per ton of sludge solids processed is approximately $54.26.

Only those costs that are easily estimated and allocated to the solids processing trains were presented above. The two remaining cost items that will be significant are electricity and maintenance (contract and by MWCC tradesmen). At the present time no attempt has been made to estimate these costs.

For those cost categories considered the dewatering and incineration cost per ton of sludge solids totals approximately $110.90. It does appear that this unit cost can be reduced by improved process control. The elimination or reduction of chemical overdose and underdose should result in a net reduction of chemicals and provide a more uniformly conditioned sludge for filtration. By manipulating the filter variables, submergence, drum speed, and vacuum, the filter cake yield can be adjusted to meet operating goals and produce a more uniform output. Uniform loading of the incinerator will significantly reduce the temperature excursions and should reduce fuel consumption. It is also anticipated that the total weight of sludge solids processed can be increased by maintaining steady operation. If a significant change can be obtained the unit cost for labor may be reduced; however, there will most likely be a labor cost associated with improved process control in terms of process control hardware maintenance.

CHEMICAL CONDITIONING

A. Preliminary Studies

Chemical conditioning is used to alter the dewatering characteristics of sludges. In order to achieve control over this operation it is necessary to estimate or directly measure the dewatering characteristics of the sludge as a function of chemical addition. The two most commonly used methods of estimating sludge dewaterability on a filter are capillary suction time (CST) and specific resistance (SR). The CST method developed by Baskerville and Gale (3) involves the time measurement of water withdrawn by gravity from a sludge sample placed in a metal cylinder. The rate of water movement is measured by the time required for the water to move through a standard distance on a section of chromatography paper. CST measurements have been related to the chemical dose applied to sludge, to the resulting pH (4), and to characterize the filterability of conditioned sludge exposed to shear forces (5). The SR method of determining sludge filterability developed by Coackley (6) involves filtration of a conditioned sludge sample through a given filter medium under a known vacuum. The volume of filtrate as a function of time is used to calculate the SR. This parameter has been related to chemical dose, pH and CST (4), (7), and vacuum filter performance (8).

Ideally as the sludge dewatering characteristic changes an on-line measurement would send a signal which could be used to vary the chemical dose so as to achieve the desired dewatering characteristics. Unfortunately neither CST or SR are directly measurable on-line. However, it was felt that a correlation was possible between other parameters which could be measured on-line and CST or SR or both.

In order to develop these correlations previous records of the performance of the vacuum filter, chemical dosing, and pH measurements were made. These indicated some fair correlations between CST and these other parameters. The literature indicated that in addition shear stress (9) and Oxidation Reduction Potential (ORP) (10) should also be considered.

An extensive series of bench scale tests were conducted from November 1978 thru February 1979 in which samples of raw sludge were conditioned with ferric chloride and lime and measurements of pH, ORP, suspended solids, shear stress, CST, and SR were made. Table 1 gives the parameters measured. Table 1A in the appendix presents the data collected in this phase of the study. The data analyses was conducted with the Statistical Package For the Social Sciences software at the University of Minnesota Computer Center. These data yielded the following correlation equation for the SR of the sludge after ferric and lime conditioning. The correlation coefficient (R) was 0.85.

$$\begin{aligned} \mathrm{SRL} = {} & -5.9 + 0.76\,\mathrm{CSTL} - 1.52\,\mathrm{SSL} + 0.165\,\tau\ 60 - 0.098\,\tau\ 30 \\ & +0.003\,\mathrm{ORPL} - 0.055\,\tau\ 12 + 0.83\,\mathrm{PHL} - 0.03\,\mathrm{TL} \end{aligned} \tag{1}$$

The regression analyses indicated that essentially all of the useful information is incorporated into the model after the addition of the third term τ 60.

TABLE 1. PARAMETERS MEASURED IN DEVELOPMENT OF SLUDGE DEWATERABILITY CORRELATIONS

Symbol	Definition
SSR	Suspended solids concentration in raw sludge, % solids
PHR	pH of raw sludge
PHF	pH of ferric chloride conditioned sludge
PHL	pH of lime conditioned sludge
ORPR	ORP of raw sludge, mv
ORPF	ORP of ferric chloride conditioned sludge, mv
ORPL	ORP of lime conditioned sludge, mv
τ 12	Shear stress of lime conditioned sludge, 12 RPM, dyne/sq. cm.
τ 30	Shear stress of lime conditioned sludge, 30 RPM, dyne/sq. cm.
τ 60	Shear stress of lime conditioned sludge, 60 RPM, dyne/sq. cm.
CSTR	CST of raw sludge
CSTF	CST of ferric chloride conditioned sludge
CSTL	CST of lime conditioned sludge
SRL	Specific resistance of lime conditioned sludge X 10^{-11}m/kg
SSL	Suspended solids concentration in lime conditioned sludge, % solids
TL	Temperature of lime conditioned sludge, ^{o}C
PF	Ferric chloride feed % by weight dry sludge solids
PL	Lime feed % by weight of dry sludge solids

A simpler equation using the important parameters of equation 1 is given in equation 2. The latter equation has an R of 0.82.

$$SRL = 5.4 + 3.94\,(CSTL/SSL) + 0.56\,\tau\ 60 \qquad (2)$$

Unfortunately all the parameters in equations (1) and (2) are not easy to monitor on-line. For example CST cannot be monitored on-line and it is not feasible to measure shear stress at more than one value of speed of rotation of a viscometer. The best equation using parameters which can readily be measured on-line is equation 3 below with R=0.69.

$$\begin{aligned} SRL = {} & 17.2 - 1.06\,SSL + 3.41\,PHF - 3.86\,PHR + 0.006\,PHF \\ & + 0.1\,TL - 0.45\,PHL - 0.016\,\tau\ 30 \end{aligned} \qquad (3)$$

This preliminary analysis indicated the potential feasibility of constructing a workable sludge conditioning model from on-line data.

B. On-line Data Collection and Instrumentation

In order to verify or modify the correlations given above, sensors were installed in the actual chemical conditioning system to collect data on-line. First the experiences with the instrumentation are discussed below.

1. Suspended Solids

A Biospherics Models 52-H solids analyzer had been installed in June, 1977 to moniter raw sludge solids input to the vacuum filter. Calibration data collected during 1978 showed a correlation of 0.93 between the analyzer readings and laboratory analysis. The unit failed in January, 1979 and was replaced by another Model 52-H analyzer in February, 1979. Calibration data for the second analyzer also averaged 0.93 with a range of 0.87 to 0.96. These analysers have shown excellent performance.

2. pH Analyzers

Sampling systems were initially installed to transfer both raw and ferric chloride conditioned sludge to open vessels where immersion type pH assemblies were used to monitor pH. An automatic clean water flushing was installed to prevent plugging on the suction side of the sampling pumps. The suction line was kept clear; but, the suction port of the sample pumps plugged repeatedly and the sampling system was abandoned.

Insertion type pH probes were procured and installed to monitor both the raw and ferric chloride conditioned sludge. Some problems have been encountered with debris on the ferric sludge electrode and the seals between the electrode and probe housing have failed on several occasions. The subsequent leakage to the transmitter enclosure caused problems. The data collected in 1978 yield correlation coefficients of 0.89 and 0.94 for the raw ferric chloride sludge monitors, respectively. The correlation coefficients are based on comparisons of the readings of the monitors and a reference instrument.

Additional data collected over a 6 month span in 1979-80 for the raw pH probe showed that the instrument calibration was adjusted approximately once every 2 weeks. In 45 checks against pH buffer the average difference in the pH reading from the buffer standard was 0.06 pH unit. The pH probe monitoring ferric chloride sludge was recalibrated approximately once per week. In 57 checks against buffer standards the average difference in the pH reading from the buffer standards was 0.16 pH unit.

The pH of the lime conditioned sludge was monitored with an immersion type assembly inserted into the conditioning tank. Scaling of the probe and rag accumulation presented a maintenance problem. In order to maintain an acceptable response the probe had to be cleaned at least once per day. The insertion of the probe along the wall of the tank was found to create a "dead" spot in the mixing which resulted in slow response times. Better response has been achieved by mounting the probe directly into the stream existing the conditioning tank. In 1978, calibration data for the lime sludge pH probe yielded a correlation coefficient of 0.97 between instrument readings and reference values in the pH range of 12.0 to 12.4. In 1979-80 calibration data collected over a 6 month time span showed that the instrument calibration was adjusted an average of once per week. In 46 checks against buffer standards the average difference in the pH reading from the buffers was 0.15 pH unit.

3. Oxidation Reduction

ORP insertion type probes were mounted in the sludge piping to monitor the ORP in the raw and ferric chloride conditioned sludge. Calibration of the probe monitoring the raw sludge was difficult because continued drifting in potential was observed when standard millivolt solutions were used to calibrate the instrument in the -100 to -500 millivolt range. The stability problem was partially resolved by using freshly prepared calibration solutions. Forty-eight to 72 hours were required for electrode equilibration following cleaning or calibration.

The ORP probe monitoring the ferric chloride sludge was calibrated in the -200 to +200 millivolt range using stable redox standards. Ferric coating of the electrode was frequently encountered necessitating daily checking and/or cleaning.

4. Viscosity

Based on the relationships obtained between sludge filterability and laboratory viscosity measurements, an on-line viscometer (Brookfield Model VTPV, 0-500 centipoise) was installed in the sludge conditioning tank in January, 1980. The instrument was installed with a 48 inch baffle tube housing a motorized spindle suspended at the bottom of the tube. An outlet port in the tube allowed conditioned sludge to exit through the side of the tube allowing a continuous flow to pass the spindle. The bottom of the baffle tube was screened to prevent debris from entering the chamber. Initial on-line tests were conducted comparing the instrument output to CST measurements taken on the conditioned sludge exiting the conditioning tank. Despite repeated testing, the instrument output could not be correlated to the CST measurements. The viscometer was removed from the conditioning tank in order to conduct static tests in quiescent samples.

Based on the instrument output in the static tests, it appears that the viscometer is not sufficiently sensitive to changes in CST to provide useful on-line conditioning information. This must be investigated further because viscosity terms appear not only in CST correlations but in SRL correlations equations 1, 2 and 3.

5. On-Line Data and Analysis

During the period September-December 1979 a total of 65 data sets were collected with the on-line instrumentation discussed above. These data are presented in Table 2A in the appendix. Measurements of viscosity were at 30 rpm and were conducted off line, as were measurements of CST and SR. Preliminary analyses of the data did not include viscosity data. In addition it was decided to use a split model with CST correlated against parameters measured after ferric chloride addition and SR correlated against parameter measurements after lime conditioning.

Two equations presented below have been elucidated for CST by multiple correlation.

$$\text{CST 1} = 21.7-10.2(\text{PHD})+7.6(\text{PHF})+1.8(\text{SSR})-0.083(\text{ORPD})+0.064(\text{ORPF}) \quad (4)$$

$$\text{CST 2} = 60.1-1.16(\text{PHD})-3.9(\text{PF})-0.153(\text{ORPD})+1.9(\text{SSR})+10.3(\text{PHF}) +0.067(\text{ORPF}) \quad (5)$$

In these equations $\text{PHD} = \text{PHR}-\text{PHF}$, $\text{ORPD} = |\text{ORPR}-\text{ORPF}|$

$R = 0.62$ for equation (4)
$R = 0.85$ for equation (5)

Figure 4 presents a plot of these two equations against some recent (February 1980) data. Equation (4) CST 1 fits the data better in the early part of the run while equation (5) CST 2 fits better during the latter part. Data collection will be continued to improve this correlation.

Equations 6 and 7 below were developed from the on-line data.

$$\text{SR 6} = 21.1-0.22(\text{PL})-0.3(\text{PF})-1.06(\text{SSR})+.003(\text{ORPD}) + 0.79(\text{PHD})-0.61\text{PHL} \quad (6)$$

$$\text{SR 7} = 32.4-0.29\text{PF}-1.04\text{SSR}-1.79(\text{PHL}) \quad (7)$$

where ORPD and PHD are the same as in equation (4) and (5).

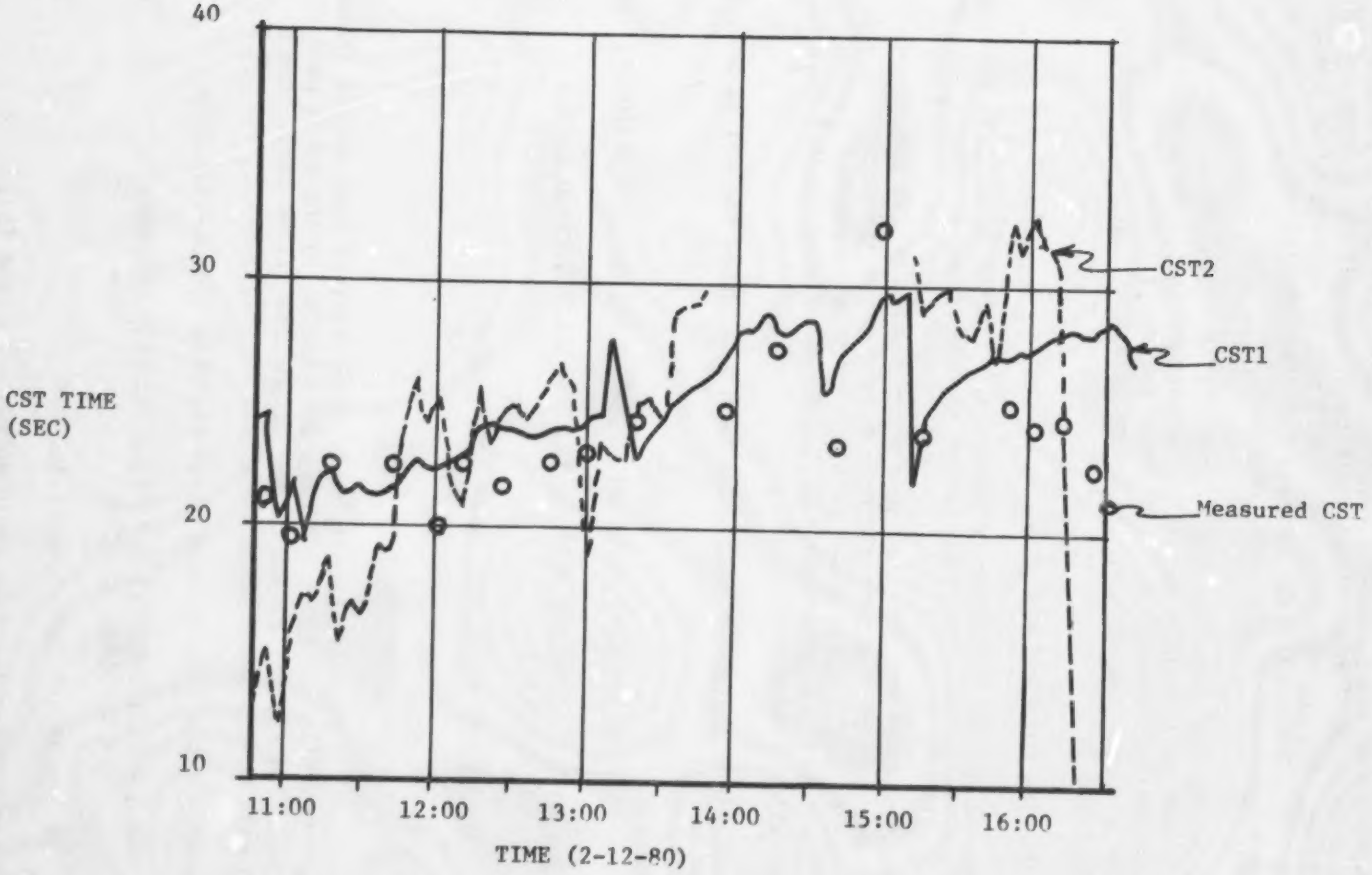


FIGURE 4. COMPARISON BETWEEN CST PREDICTED BY CORRELATION EQUATIONS FROM ON-LINE SENSOR DATA AND LABORATORY MEASUREMENT

Equation 6 has the better correlation coefficient R=0.8 than equation 7 R=0.67. The former equation contains more terms and a greater number which occur in the CST correlation equations 4 and 5. Data collection will continue to improve these equations. It is hoped that viscosity measurements can be included as these should improve the equations developed. In addition some consideration is being given to the use of a non-linear equation format to fit the data. Finally SR values directly generated by the full scale filter operation rather than by laboratory tests will be available for the final correlation.

C. Design of Chemical Conditioning Control Systems

1. Control Loop Decoupling

The conditioning control problem is to design a control strategy that controls the ferric chloride addition and the lime addition by adjusting pump speeds so that the sludge is conditioned properly. Tests conducted early in the project indicated that for optimal conditioning the two chemicals should be added at different locations in the process flow. This would avoid the ferric and lime interacting with each other instead of the sludge. These tests resulted in changing the ferric chloride addition point from just below the mix tank to just after the magnetic flow meter in the sludge feed line. With this change the ferric chloride had about 2 minutes (100 feet of pipe) to condition the sludge prior to reaching the lime addition point. A second result of this separation was that the control strategy was decoupled and made serial; first ferric, then lime, with a two minute time difference. Hence, the conditioning control strategy was broken into two control loops for design.

Design of the conditioning control systems was conducted prior to and coincident with the development of the chemical conditioning models presented previously. In the discussion below some of the activities in conjunction with control based on simpler models (i.e. mass flow proportional) than those which were developed is discussed. It was this early work which partially indicated that more sophisticated conditioning models would be required if the system was to be optimized.

2. Ferric Chloride Control

The first ferric chloride control strategy was a simple feed forward mass flow pacing control. A ratio was entered and the ferric chloride metering pump speed was adjusted in proportion to the pounds of solids entering the process. This strategy functioned as designed but resulted in periodic over and under dosing whenever sludge type or quality changed. Because of the variability of sludge quality, it became very difficult to determine the correct dose ratio at any given time without extensive lab analysis.

In order to minimize the effects of sludge variability on the control, a feed back type of control was designed. The most significant measurable feedback variable as determined by lab conditioning tests was pH. The requirement that the variable be measurable by on-line instrumentation heavily influenced this selection. The first implementations of control using a pH loop were unsuccessful due to instrument/sensor problems. The pH sensor depended on a sample system that proved to be unreliable. However, the experiments did indicate that the pH control approach would work. The sampling system was removed and in-line probes were installed to measure both pH and ORP of the ferric conditioned sludge.

Testing was conducted to determine the process dynamics for a pH control loop. Parallel tests were conducted simultaneously on ORP as a second feed back variable. The ferric tests show that both the pH probe and the ORP probe measure a change when the ferric pump speed is changed. The pH signal was observed to contain much more noise than the ORP signal. The frequency of the pH noise appears to correspond to the stroke rate of the ferric pump, which indicates that mixing is not complete in the sludge line. The ferric conditioner may be remaining in pockets in the sludge line at regular intervals thereby causing the signal "noise." The ORP signal is relatively smooth.

The ferric tests Table 2 indicate that the time before response (deadtime) of the pH and ORP are about the same. However, the process storage time (time to reach 63% of final value) for the pH signal is much shorter than the ORP. Since the pH signal would require a filter (which introduces a delay), the pH and ORP signals are relatively equivalent, dynamically, in their advantages and disadvantages for use as feedback in the control.

The analyses of the $FeCL_3$ testing provides the prerequisite data to construct a dynamic model for pH of the process. A block diagram of this model is shown in Figure 5. The model is expressed in Laplace transform notation for control system convenience. The model indicates that a change of 1 gpm in pump speed multiplied by the process gain of -0.89 would result in a pH change of -0.89 after a sufficient time was elapsed. The deadtime and lag times appear as coefficients of the Laplace variable S. Using this dynamic process model the theoretical gains for the PID three mode controller were calculated.

Even with a closed loop pH control running, it became apparent from data on the feed sludge, that there was again no simple pH setpoint that the control loop could use. Thus, the feed back approach; even though variations in pumping and in ferric chloride concentrations were now eliminated, required additional parameters to make the loop functional and responsive to real sludge conditioning needs.

As indicated previously the best indicator of how well the sludge had been conditioned after ferric chloride addition was determined to be either a (SR) analysis or a (CST) measurement. However, neither of these parameters were measureable with instrumentation for use in a control system. Both, however, were parameters that could be setpointed with a specific value with a reasonable degree of confidence. A control strategy that could control the CST of the ferric conditioned sludge so that the CST stayed within a relatively small deadband around a stated setpoint should in principle compensate for normal changes in sludge quality. CST was chosen over SR because it was a

TABLE 2. PROCESS DYNAMICS $FeCl_3$ CONDITIONING

				pH			ORP		
Date	Time	Chart	pump change (GPM)	K process gain	TD dead time (min)	TPS storage time (min)	K	TD (min)	TPS (min)
9-19	11:30	A	2.6	-1.00	1.5	2.0	-1.10	1.4	3.2
9-19	13:50	B	2.4	-0.74	1.8	2.3	-0.72	1.2	3.4
9-19	14:27	C	2.7	-0.80	1.0	2.2	-0.70	1.4	4.7
9-21	15:14	D	2.7	-1.03	1.3	1.8	-1.29	1.3	3.6
			Averages	-0.89	1.3	2.1	-0.95	1.3	3.7

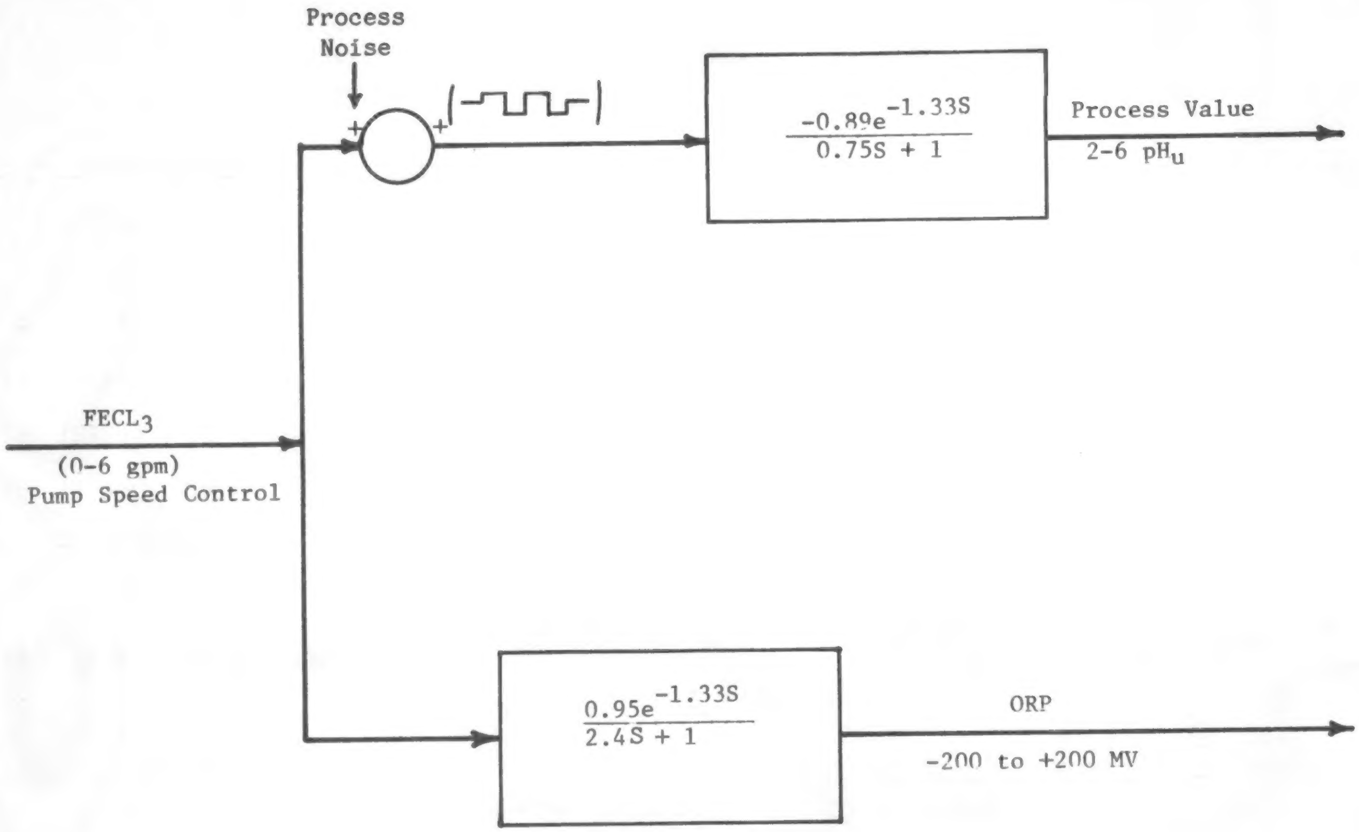

Process Noise
+
+
-0.89e^-1.33S / 0.75S + 1
Process Value
2-6 pH_u
FECL_3
(0-6 gpm)
Pump Speed Control
0.95e^-1.33S / 2.4S + 1
ORP
-200 to +200 MV

FIGURE 5. DYNAMIC MODEL FOR $FeCl_3$ EFFECT ON pH AND ORP OF SLUDGE

much simpler measurement to make and correlated very well with specific resistance when solids were taken into account.

The previous sections have discussed in detail the development of a model that relates measureable process parameters to the ferric conditioned sludge CST. The use of this model in the control strategy design is shown in Figure 6. The pH feedback loop is retained because of its advantages of 1) providing corrective action on errors, 2) eliminating pump metering fluctuations, 3) eliminating ferric chloride concentration fluctuations, and 4) minimizing the dependence on process sensors.

The model receives inputs from the process sensors and calculates a predicted conditioned sludge CST value. This prediction is then compared to the desired CST setpoint. If they are different an adjustment is calculated for the pH controller's setpoint which ultimately will change the CST to its desired value. To avoid coupling problems the control part of the CST model is designed to be recalculated very infrequently relative to the pH controller, thereby appearing to be constant to the controller. The control strategy flow chart is shown in Figure 7.

3. Lime Control

The development of the lime control strategy paralleled the ferric control very closely. First a mass flow proportional control was designed, followed by a pH feed back control which led to a modeling effort. In this case the SR of the lime conditioned sludge was chosen because it can be related to filter yield by a procedure described in the next section. The control strategy for lime is shown in Figure 8. Either equation 6 or 7 or some variant will be used as the SR model.

Dynamic testing of the lime pH response was conducted to determine the process model and the 3-mode controller tuning constants. The lime tests show that the pH after lime addition is very high and relatively insensitive. Large changes in lime pump speed were necessary to obtain relatively small changes in pH. In some of the tests lime dose was so high at the beginning of the test that virtually no change was observed in the pH.

The dynamic model based on this testing can be expressed in Laplace Transform Notation as:

$$\underset{(0.15\text{ gpm})}{\text{CaO}} \longrightarrow \boxed{\frac{0.44\ e^{-3.15S}}{4.2\ S + 1}} \longrightarrow \underset{(9\text{-}13\text{ pH})}{\text{pH}}$$

Therefore a change of 1 gpm will result in a pH change of 0.44 after a sufficient time period and provided the pH is not already near the saturation point.

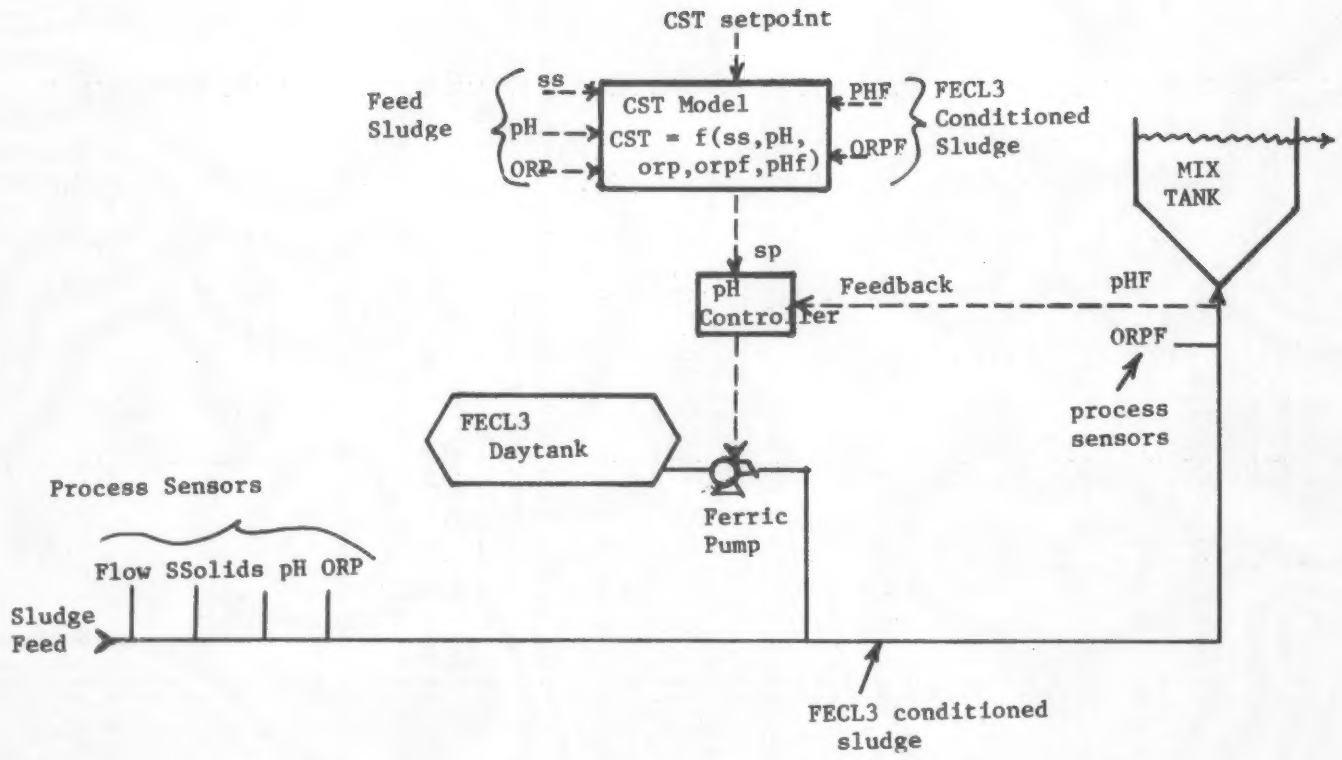

CST setpoint
Feed Sludge
ss
pH
ORP
CST Model
CST = f(ss,pH, orp,orpf,pHf)
PHF
ORPF
FECL3 Conditioned Sludge
sp
pH Controller
Feedback
MIX TANK
pHF
ORPF
process sensors
FECL3 Daytank
Ferric Pump
Process Sensors
Flow SSolids pH ORP
Sludge Feed
FECL3 conditioned sludge

FIGURE 6. FERRIC CHLORIDE DOSE CONTROL

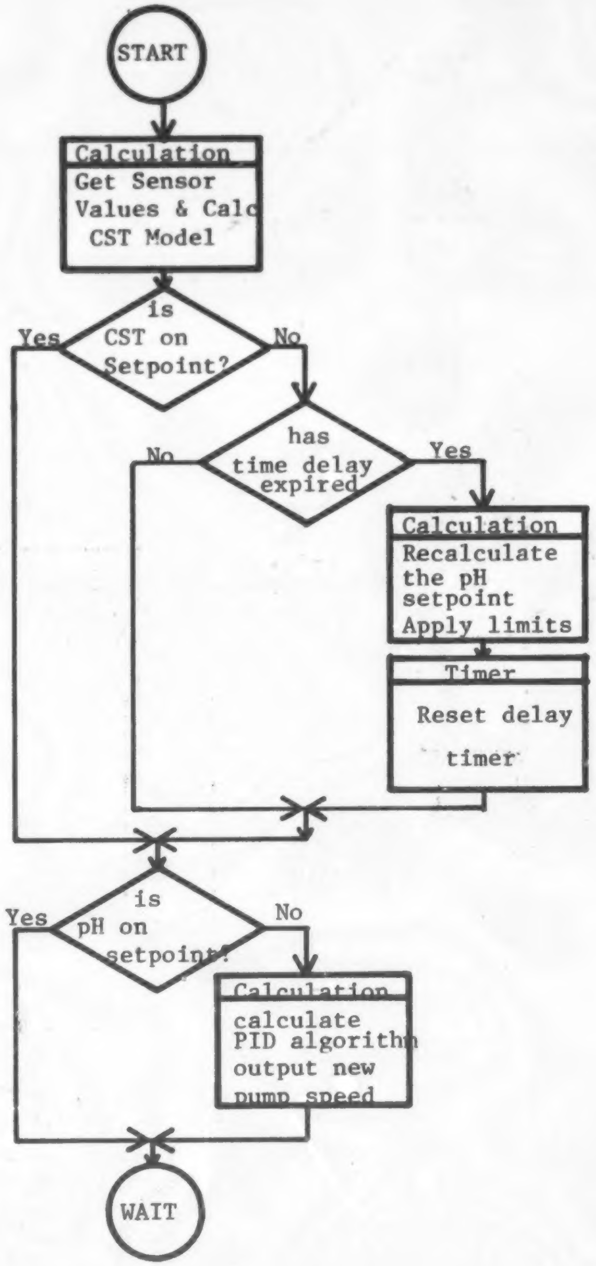

START
Calculation
Get Sensor Values & Calc CST Model
is CST on Setpoint?
Yes
No
has time delay expired
No
Yes
Calculation
Recalculate the pH setpoint Apply limits
Timer
Reset delay timer
is pH on setpoint?
Yes
No
Calculation
calculate PID algorithm output new pump speed
WAIT

FIGURE 7. CST CONTROL STRATEGY FLOW CHART

TABLE 3. PROCESS DYNAMICS LIME CONDITIONING

Date	Time	Pump Change gpm	K	TD (min)	TPS (min)
9/14	1130	2.05	-	-	-
9/21	1451	3.2	.33	3.0	9.3
9/21	1558	14%	.54	3.3	5.4
9/26	1704	9%	-	-	-
10/2	1604	2%	-	-	-
		Averages	.44	3.15	7.35

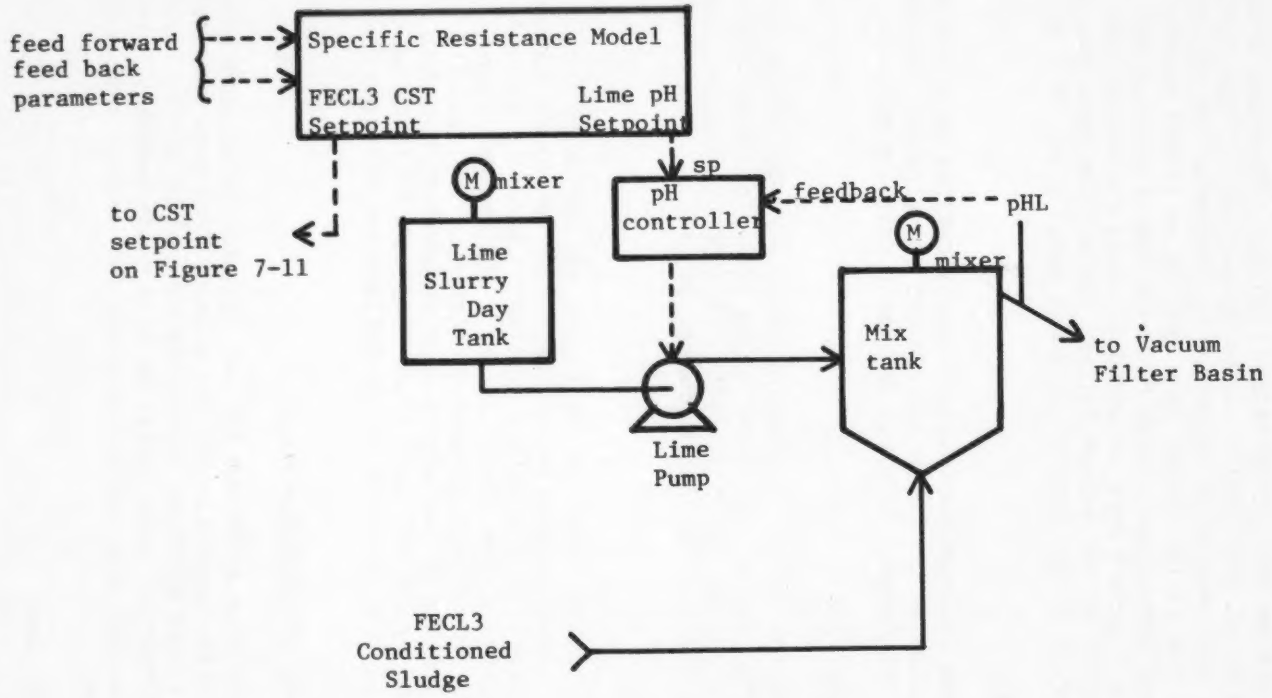

feed forward
feed back
parameters
Specific Resistance Model
FECL3 CST
Setpoint
Lime pH
Setpoint
to CST
setpoint
on Figure 7-11
M
mixer
Lime
Slurry
Day
Tank
sp
pH
controller
feedback
pHL
M
mixer
Mix
tank
to Vacuum
Filter Basin
Lime
Pump
FECL3
Conditioned
Sludge

FIGURE 8. LIME DOSE CONTROL

DEVELOPMENT OF VACUUM FILTER CONTROL

A. Preliminary Work

Preliminary work on the vacuum filter included literature studies, field observation and lab test data to identify possible filter control strategies and the instrumentation necessary to implement the strategies. Manual operation of the filter relies very heavily on visual observation and experience for proper operation. Based on these investigations the instruments shown in Figure 9 were selected to study the automation and optimization of control on the vacuum filter. As will be seen this selection was based on the vacuum filter control strategy model given in C below.

B. Instrumentation

For each of the instruments involved specifications were prepared to ensure proper operation in the harsh environment anticipated. Below a brief description of the instruments is given and their relative success.

1. Raw Sludge Flow

Sludge flow to the filter is measured using a Fisher-Porter magnetic flowmeter. This meter is part of the existing plant instrumentation installed when the plant was built in 1968. This meter provided a reliable metering of the raw sludge with little upkeep required.

2. Vat Level

Measurement of sludge level in the filter vat was added as part of the automation project. Manual operation of the filter relies on periodic visual inspections of the level. Satisfactory instrument accuracy was achieved with both sonic and capacitance type level sensors. The Endress Hauser sonic transmitter was utilized for the control strategies. The use of a stilling well and digital filtering of the signal were required to minimize the effect of wave action caused by the vat agitation. Frequent flushing of the stilling well was required to avoid sludge buildup and maintain instrument accuracy.

3. Drum Speed and Agitator Speed

Both the filter drum drive and the vat agitator drive were originally equipped with hand crank speed controllers without feed back ability. Both variable drive units were modified for remote control by adding reversing motor actuators; tachometers were installed to provide feedback. These modifications provide the interfacing necessary for control with a computer based control system.

4. Vacuum Pressure

Form vacuum and dry vacuum measurements were originally made using local gauges. Control was implemented through manual valves. Rosemount differential pressure transmitters were installed on both the form and dry

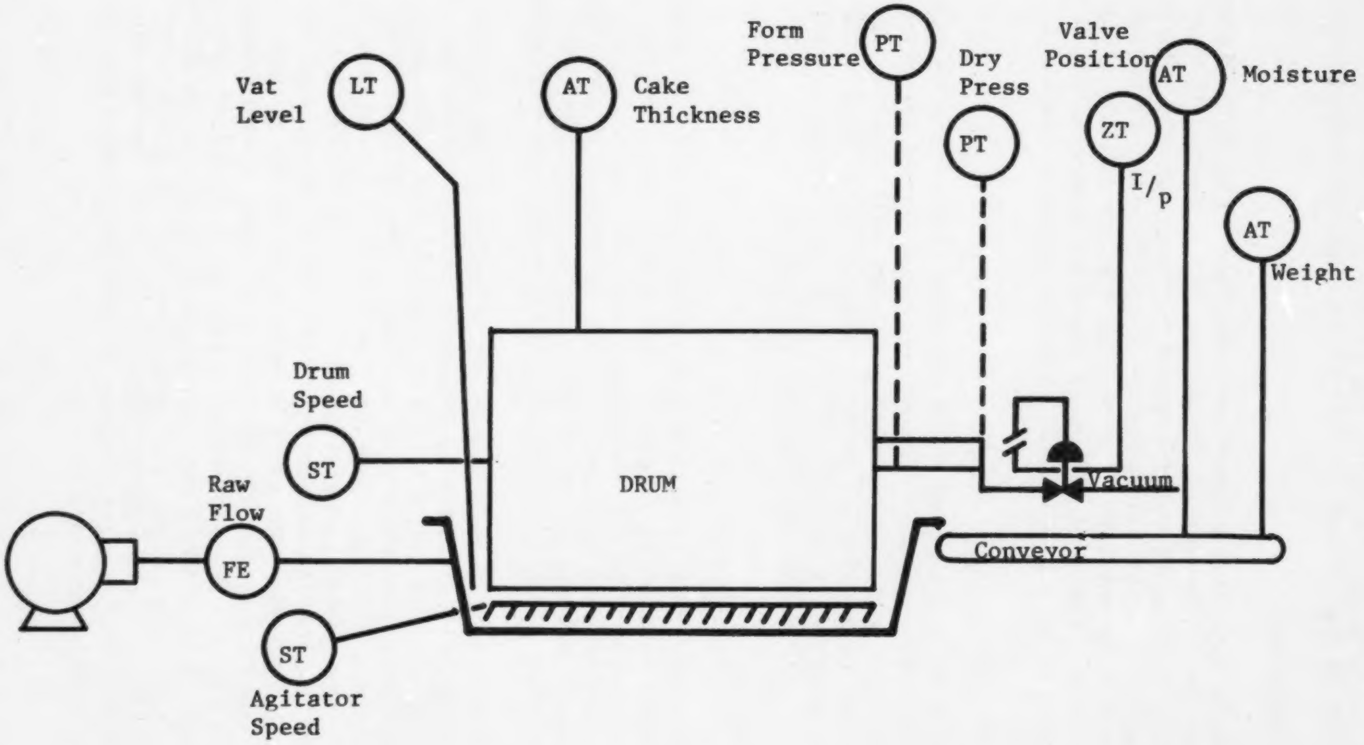

Vat Level
LT
AT
Cake Thickness
Form Pressure
PT
Dry Press
PT
Valve Position
ZT
I/p
AT
Moisture
AT
Weight
Drum Speed
ST
DRUM
Vacuum
Raw Flow
FE
Conveyor
ST
Agitator Speed

FIGURE 9. INSTRUMENTATION OF VACUUM FILTER

vacuum lines for remote monitoring. A motor actuated centerline valve was installed on the form vacuum replacing the existing manual valve. The automatic valve can be controlled locally or through the computer remotely.

5. Cake Thickness

A Wesmar sonic type transmitter was installed to measure the thickness of the cake formed on filter drum. The sensor is calibrated to read 0-1" of cake. The modified instrument range created difficulties but yielded satisfactory results. The signal required heavy filtering to smooth the process noise created by the variation in cake surface and the slightly eccentric filter drum. The drum is out of round by approximately 0.2 inches. Zero drift is somewhat of a problem and requires frequent checking.

6. Cake Weight

Filter output is measured by a BIF conveyor scale. The filter was originally equipped with a scale which was replaced during the automation study. Irregularities in cake placement along the belt require the instrument output to be heavily filtered. Zero shift has been a continuous problem with the instrument, most of which can be attributed to water on the belt or buildup of sludge cake on the conveyor system.

7. Cake Moisture

On-line measurement of filter cake moisture content is an important feedback measurement indicating vacuum filter performance. Because of the importance of the measurement for automation and the lack of tested instruments for on-line cake moisture analysis, a testing program was undertaken. Several instruments, from different manufacturers were tested that utilized infrared reflectance and microwave reflectance techniques.

The performance of the instruments was affected by the uneven loading of cake on the conveyor belt, varying distances between sensor and material on the belt, sludge surface characteristics, and in some cases sensitivity to the residual conditioning chemicals in the cake. The tests indicated that the moisture analyzers did track together, even though variations in sludge cake orientation, etc., made correlations to actual moisture look very poor. Tests of analyzers in the lab measuring mixed samples correlated much better to actual moisture than when the units were mounted on the conveyor. Based on this fact a sampling system was devised to deliver a more homogenous sample to the analyzers. The sampler consisted of a pump that ground up the cake and delivered it to a small conveyor where moisture was measured. The results of this test were mixed, and compounded by the problems of sampling the cake. The testing program results indicate that the instruments tested to date are not suitable for the application of measuring cake moisture on a vacuum filter discharge conveyor.

C. Modeling

The vacuum filter model which will be used is based on the yield equation given below. This equation is derived from mass balance considerations (1).

$$L = \left[\frac{2\,P \quad w\,X}{u\,(SR)\,t}\right]^{1/2} \qquad (8)$$

L = Filter Yield Dry Solids/Unit Area/Unit Time
P = Vacuum Level in Form Zone
w = Dry Cake Solids per Unit of Filtrate
x = Ratio of Form Time to Cycle Time
u = Viscosity of Filtrate
t = Time of 1 Filter Cycle
SR = Specific Resistance

It can be seen that the parameters in this equation with the exception of SR are physical constants of nature (u), physical operating parameters of the filter (P, x, t) or output goals (L, w). Thus if the correlations which are being established in this study between SR and on-line parameters (pH, ORP, etc.) are satisfactory, feed forward control of vacuum filter operation will be achieved. As indicated previously tests are continuing to sharpen the correlation equation for SR. Especially important is to use data from full scale operation of the filter to determine values of SR for the correlation. Figure 10 presents some data collected in 1978 with SR values determined in the laboratory and SR values calculated from the vacuum filter. These data indicate fair correlation between SR values generated in the laboratory and SR values calculated from filter performance, and reinforce the need to establish the final correlation using full scale filter generated SR values.

D. Proposed Control System Design

Control of the vacuum filter may be broken down into four control loops, corresponding to the four parameters which may be varied. They are vat level, drum speed, pressure and agitation speed. All but the agitator speed are included as parameters in the yield equation above and their relationship to production may be determined using the yield model. A description of each of the proposed control loops is described below.

1. Vat Level

This control loop (Figure 11) controls the speed of the raw sludge pump to maintain the vat at a set level. The algorithm is a cascade control loop. The internal loop is a flow control loop which receives the sludge flow signal as feedback and outputs a speed adjustment signal to the pump motor. The outside cascade loop provides the flow setpoint to the flow controller based on vat level.

The level setpoint to the outside loop can be operator input or controlled by an algorithm which modulates the level setpoint based on a manually entered cake moisture content value.

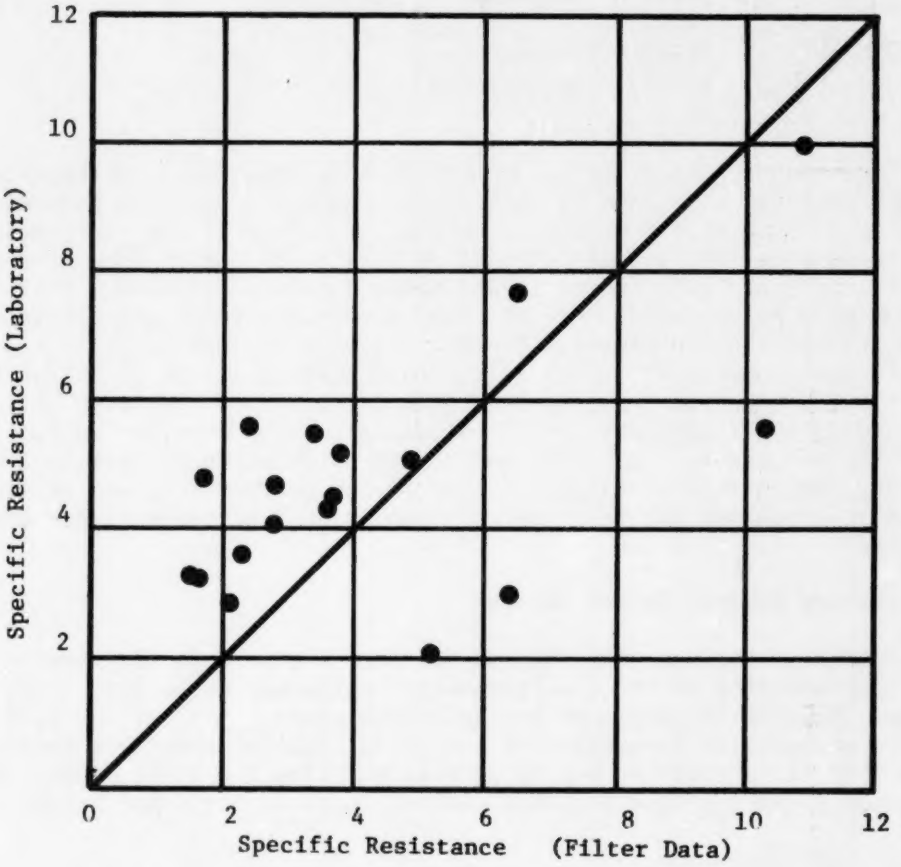


FIGURE 10. COMPARISON OF LABORATORY & VACUUM FILTER GENERATED VALUES OF SPECIFIC RESISTANCE ON THE SAME SLUDGE

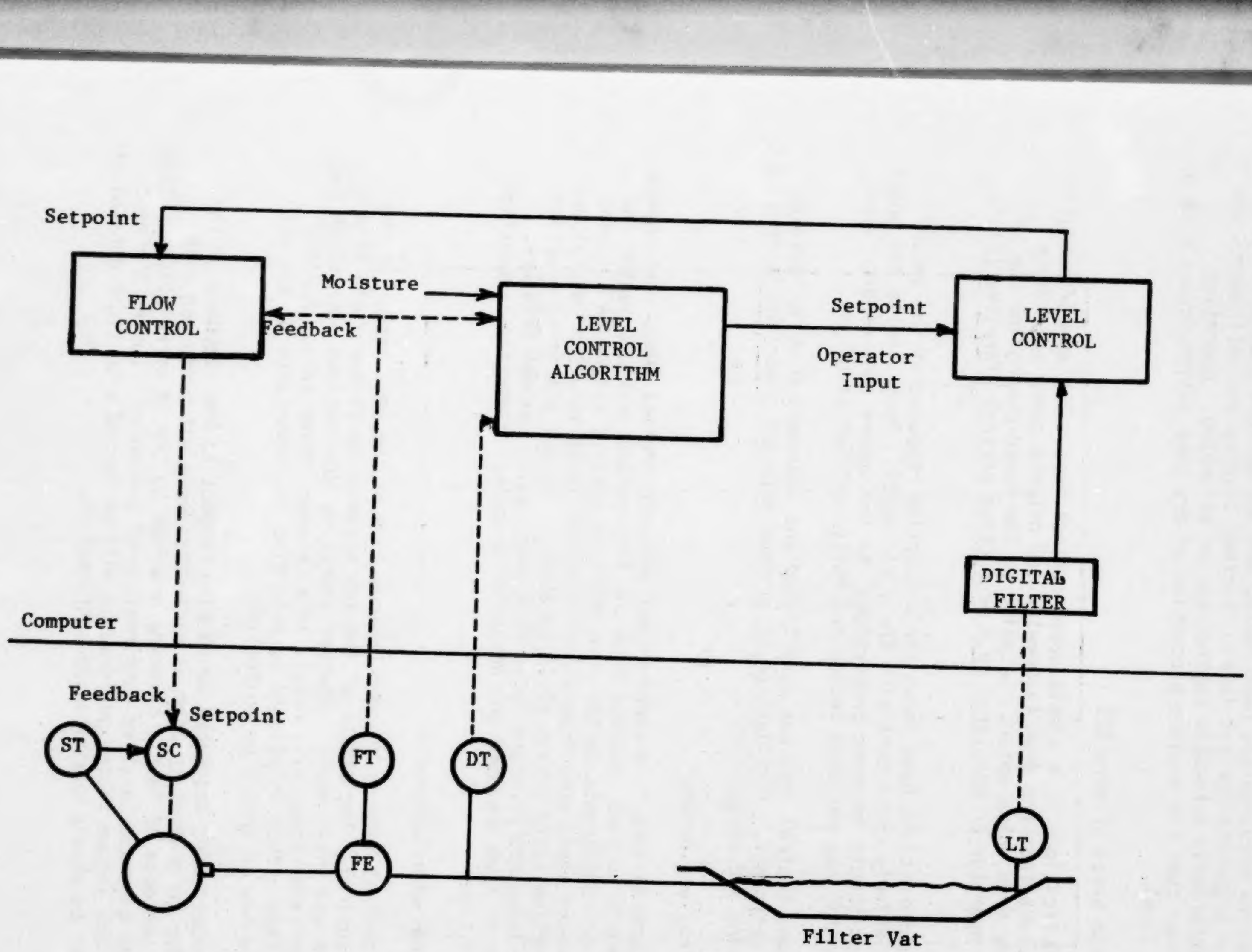

Setpoint
FLOW CONTROL
Moisture
Feedback
LEVEL CONTROL ALGORITHM
Setpoint
Operator Input
LEVEL CONTROL
DIGITAL FILTER
Computer
Feedback
Setpoint
ST
SC
FT
DT
FE
LT
Filter Vat

FIGURE 11. FILTER VAT LEVEL (DRUM SUBMERGENCE) CONTROL

This control strategy has been tested and implemented successfully using operator setpoints for vat level. Further testing and refinements are necessary before fully automatic control can be attempted. Operating experience shows that the maximum production of dry cake solids occurs with a very low vat level.

2. Drum Speed (Figure 12)

This control loop is a simple speed control loop. The speed controller receives feedback from the drum tachometer and outputs speed adjustment signals to the drum drive motor. Setpoint to the speed-controller can be input by the operator or modulated by a production control algorithm (Filter Yield).

Early observations have shown the production response of the vacuum filter qualitatively consistent with the yield model. According to the model dry production should increase proportional to the square root of drum speed. This relationship has not been tested and fully verified as of yet.

The speed control loop has been tested and implemented using operator input speed setpoints. Finalization of a speed setpoint algorithm is pending further testing and evaluation.

3. Pressure Control

This control loop is a conventional pressure control loop. The pressure controller would receive feedback from the form vacuum pressure sensor and output control adjustments to the form pressure control valve. A setpoint for the pressure would come from either operator input values or a pressure control algorithm based on the yield equation. Normal operating practice has been to leave the pressure valve full open, using maximum pressure. Optimization of form pressure and design of a control algorithm is questionable at this time.

4. Agitator Control

This control loop is a speed control loop in the same manner as the drum speed control. The effect of the vat agitator on filter yield is not addressed in the yield model. However operating observations have shown that increased agitator speed will result in a thinner, dryer cake. Further testing of this effect is planned to determine the importance of the vat agitator and how it should be controlled.

The experiences described above with respect to the response of the vacuum filter to changes in control parameters were not performed with control of the dose of ferric chloride and lime by the SR correlations. Thus they can not give quantitative confirmation of the model. During the next few months the vacuum filter performance will be tested with tight control of SR, in order to obtain quantitative confirmation.

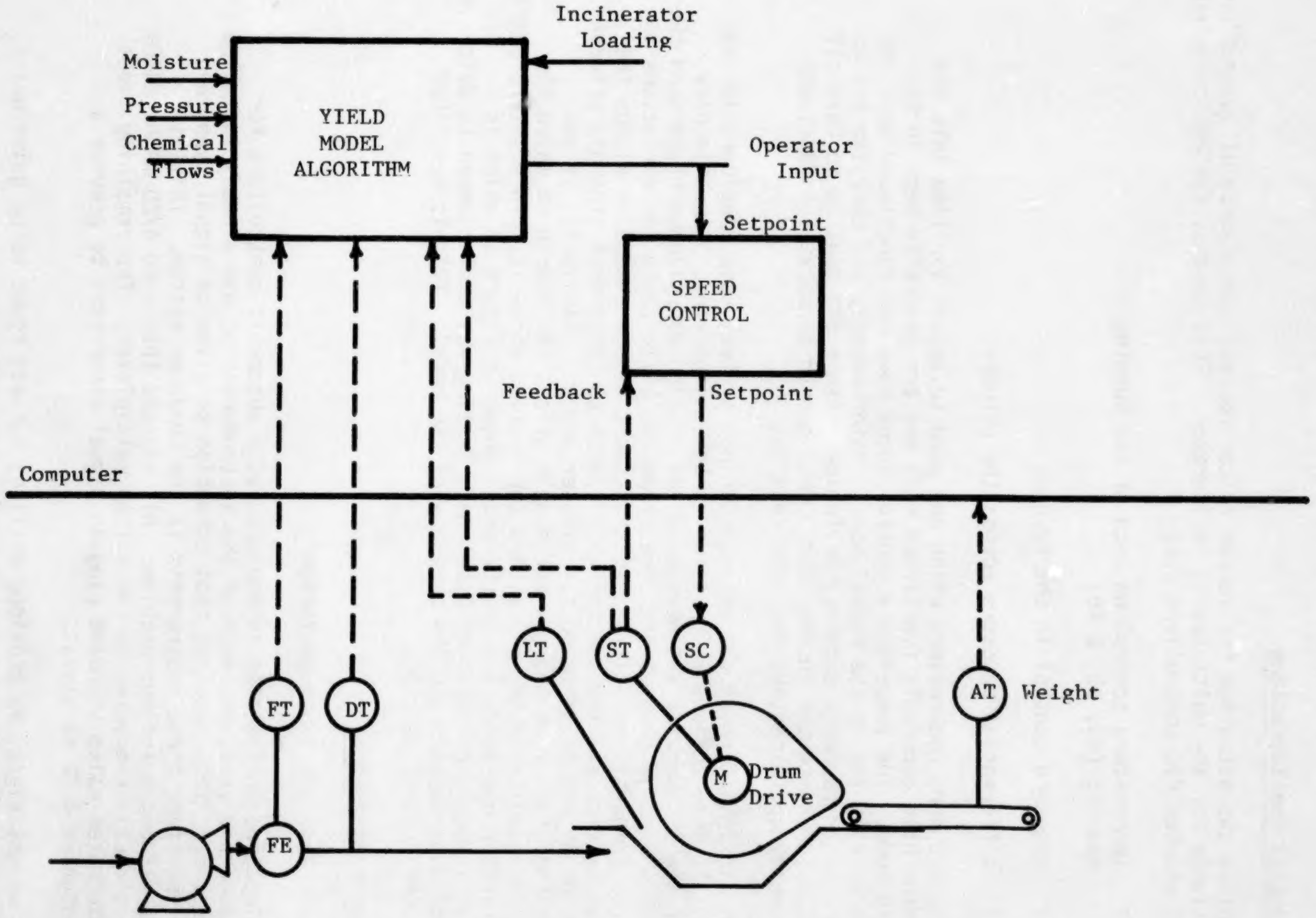

Incinerator Loading
Moisture
Pressure
Chemical Flows
YIELD MODEL ALGORITHM
Operator Input
Setpoint
SPEED CONTROL
Feedback
Setpoint
Computer
LT
ST
SC
FT
DT
AT
Weight
M
Drum Drive
FE

FIGURE 12. VACUUM FILTER CONTROL

DEVELOPMENT OF INCINERATION CONTROL

A. Initial Considerations

Unlike the situation for vacuum filter control some degree of automation was available for the multi-hearth incinerator. This control system consisted of panel mounted PID controllers for:

a) temperature control on each of the burning hearths (#3, #5, & #6)

b) pressure control in the furnace

c) differential pressure across the scrubber

The pressure controllers which are used to adjust air flow into the incinerator have generally functioned well and are generally kept in the automatic mode. The temperature control loops have not functioned well and are thus usually set in the manual mode. Unfortunately the operator has no feed forward or feedback strategy to follow, rather the goal is to burn all the sludge coming to the incinerator. Thus hearth temperatures fluctuate significantly and excessive fuel use results.

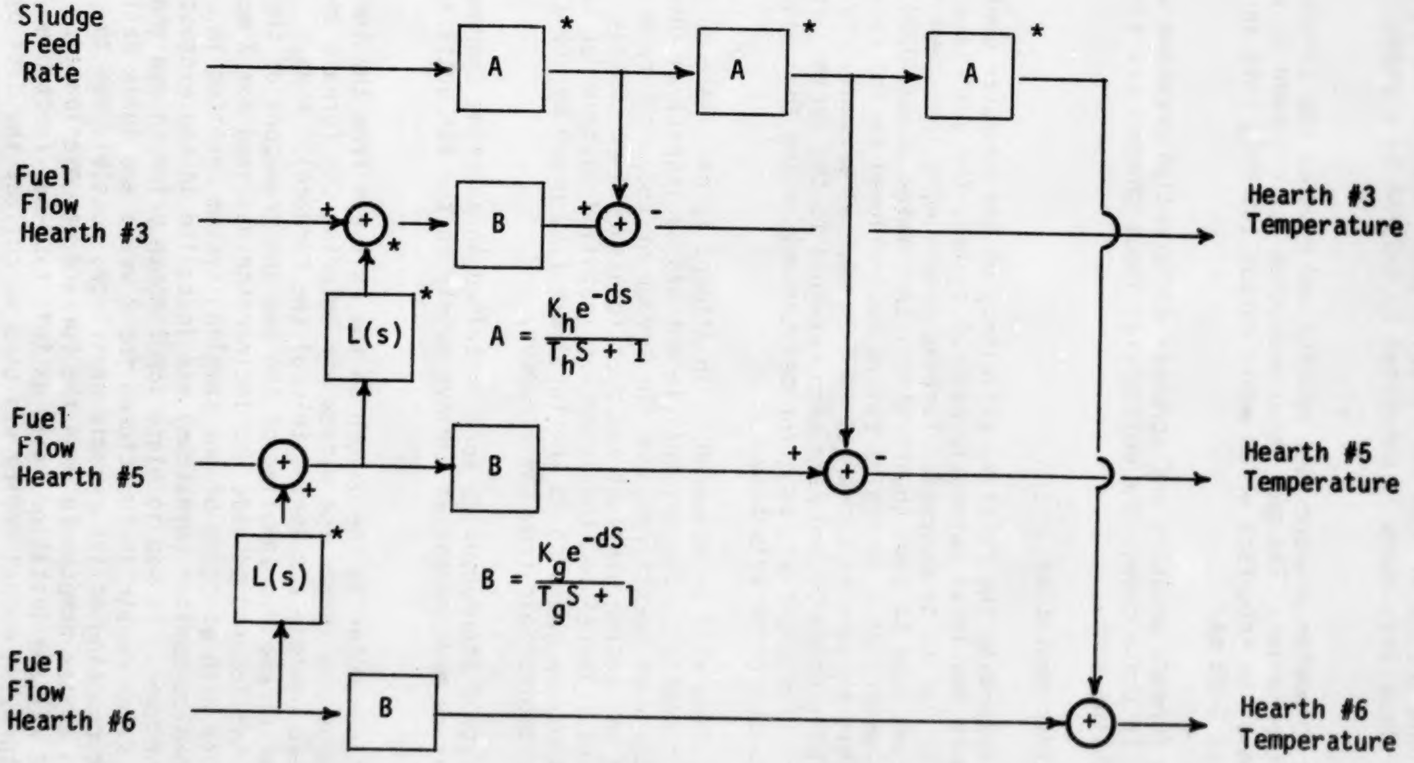
To aid in determing the optimum control system, some consideration was given to the interactions which occur in the incinerator. A preliminary model of the incinerator was developed and testing was planned based upon the model. Sludge flow changes are considered the basic cause for variations in the hearth temperatures. It seemed consistent that a change in sludge feed rate would affect all the hearths with hearth #3 being most strongly effected and hearths #5 and #6 affected to a lesser degree. Similarly, it was expected that fuel flow changes would only affect the hearth on which the change was actually taking place, and the hearths above it. This interpretation of incinerator operation led to the model shown in Figure 13 which is presented in Laplace Transform Notation. Testing was then planned to determine the process parameters and the accuracy of the model. This is described below under Dynamic Testing.

B. Instrumentation

1. Existing Instrumentation

Since the incinerator is equipped with automatic controllers for temperature and pressure, much of the instrumentation was already installed and operating. This required that attention be given to signal types and ways to interface these instruments to the computer system. This, in general, was not a serious problem. All signals that were 4/20 mA or 10/50 mA were directly connected to the system multiplexer. The remaining types were interfaced using standard plug-in signal converters to provide a representative 4-20 mA signal.

A weight signal is provided on-line, but was found to be quite noisy. Filtering was added to make this a useful parameter.


Sludge Feed Rate
Fuel Flow Hearth #3
Fuel Flow Hearth #5
Fuel Flow Hearth #6
A
B
L(s)
+
-
*
$A = \frac{K_h e^{-ds}}{T_h S + 1}$
$B = \frac{K_g e^{-dS}}{T_g S + 1}$
Hearth #3 Temperature
Hearth #5 Temperature
Hearth #6 Temperature

* Anticipated Interaction

FIGURE 13. INITIAL INCINERATOR MODEL

Damper positions are taken from the existing panel mounted indicators; the voltage across these indicators is converted to 4-20 mA by a signal converter.

The existing bolometer measures wet opacity and records the information on a circular chart recorder. The pen servo mechanism was returned to the vendor for addition of an interface which would output 1-5 VDC, this in turn will be converted to 4-20 mA.

The existing furnace pressure and scrubber differential pressure were 10-50 mA signals. In these cases, the multiplexer input channel was simply wired into the existing loop.

2. Added Instrumentation

In order to determine the relative efficiency of the computer control system as compared to the local automatic control system, the auxiliary fuel into the incinerator has to be metered. Turbine meters were installed in the main natural gas line feed to each incinerator. Each meter is equipped with a pulse head which generates a series of pulses proportional to the rate of gas being used. This in turn is converted to 4-20 mA. At present, these meters are not working properly and have been returned to the factory for repair. A similar arrangement was used for measurement of the fuel oil flow. These have not been tested at this time.

The exit gas flow will be measured in an attempt to calculate an energy balance for the incinerator. The original intent was to install an insertion turbine meter in the duct downstream from the breach of the incinerator. However, there was not enough straight run duct to assure a reasonably accurate measurement. Therefore, the meter was installed upstream of the ID fan damper. The flow now measured is the total air flow drawn by the ID fan which includes the ambient air from the scrubber.

Additional Type K thermocouples were installed in existing thermowells to provide temperature measurement at various points in the air ducts and on each hearth.

An important parameter is the oxygen in the exit gas from the incinerator. The higher this value the larger the excess air entering the furnace and the greater the fuel use (excess air serves to cool the furnace). A Hays Republic monitor which employs sampling of the gas and transport of the sample to the monitor located outside the incinerator was used for 2 months. Continual difficulty with plugging of the sampling system resulted in abandonment. A in-situ monitor (Dynatron) was installed in the exhaust duct upstream of the scrubber. It was in place for 1 month prior to use and was found defective. After repair it functioned for 2 weeks and again failed. Failure was due to cracking of its ceramic cartridge; possibly due to exposure to a zero oxygen atmosphere when the furnace was overloaded. It is being repaired and will be installed again as this type has functioned well in other incinerators. A manual device was used to collect the data discussed here.

The rabble arm speed is to be measured by a proximity switch which will detect the presence of a rabble arm drive gear tooth. This will result in a pulse train proportional to the speed of the rabble arm. As in the fuel flow measurement, the pulse train will be converted to a 4-20 mA signal for input to the multiplexer.

C. Dynamic Testing

1. Temperature Characteristics

a) Objective

In developing temperature control systems for the incinerator, a model of the temperature dynamics is necessary to determine the type and degree of interaction between the various hearth temperatures, as well as the relative speed with which process changes can occur. It is toward this end that testing of each hearth temperature dynamics was conducted.

b) Test Setup

Although computer control is intended to be implemented on Seneca Incinerator #1, testing of temperature dynamics was performed on Incinerator #2. Incinerator #1 was unavailable at the time the test was scheduled to be performed. Since the hearth construction of each is identical, it is felt that the test results are representative of each incinerator. All temperature controllers were placed in manual mode.

A thermocouple from each of hearth's #3, #5, and #6 was directly connected to a 3-pen strip chart recorder. Temperature changes were then observed on each hearth in response to various burner combustion air valve position changes. Since the burner valve positions were not connected to a strip chart recorder, the position change was simply noted on the temperature chart recording at the time of occurance. The response curve resulting from a burner valve position step change was then used to calculate the respective process gain, time constant, and deadtime.

Also tested was the hearth temperature response to variation in the shaft cooling air damper position. When the shaft cooling air damper position is 0%, all of the shaft cooling air is directed to the exhaust stack. Increasing the position to 100% increases the amount of air flow to hearth 7 of the incinerator.

c) Observations

The calculated process parameters resulting from the step-change tests of the burner combustion air valves are presented in Table 4. The deadtime values are the result of a second series of

TABLE 4. ESTIMATED MODEL PARAMETERS FOR INCINERATOR

Test No.	K_3	T_3	D_3	Comments
3	1.4	4.0	0.4	Hearth #5 is not fired.
15	1.2	3.6	0.4	

Test No.	K_5	K_{56}	D_5	T_5	Comments
5	0.8	1.9	0.2	4.0	Hearth #6 is not fired.
6	0.9	2.1	0.2	4.0	Hearth #6 is not fired.
16	2.4	6.1	0.2	4.0	

Test No.	K_6	T_6	D_6	K_{65}	T_{65}	D_{65}	Comments
7	1.3	6.0	2.5	1.9	6.0	1.0	Startup of hearth #6 from 0-50%
11	0	-	-	0.8	6.0	1.0	
12	0	-	-	1.0	4.5	0.5	
18	0.5	2.0	0.8	0.7	3.0	0.7	

Test Conditions

Furnace #2 10/13/79
Feed Rate = 4.2 tons/hour
Rabble Speed = 2.1 rpm

Legend

K = Gain
D = Dead Time
T = Storage Time

Laplace Transform $\frac{Ke^{-DS}}{TS + 1}$

tests which were performed to more accurately determine the process deadtime. These values are used in Laplace Transform format to describe the system dynamics.

During the testing, it was observed that the process gain varied considerably and cannot be considered a constant value in the process model. Figure 14 is a plot of the calculated process gain as a function of burner combustion air damper position. The process control system will have to accomodate this non-linear gain function.

The observed temperature response indicated that, as anticipated, fuel changes to hearth #3 had no effect on hearth's #5 and #6, and hearth #5 and #6 did no appear to have a major affect on hearth #3. It is thought that a change in temperature on hearth #3, as a result of burner firing on #5 and/or #6, probably does occur after a sufficient length of time, but this effect can be neglected for control purposes. Apparently, hearth #4 serves to isolate hearth #3 from the lower burning hearths. The testing did indicate a strong interaction between hearths #5 and #6. This intersect is accentuated by the relative location of the respective thermocouple which is discussed in more detail in another section.

During testing, the sludge feed rate was observed to fluctuate significantly on occasion. The effect of this on the various hearth temperatures can be seen in Figure 15. In this particular instance increases in fuel and air flow to hearth #3 was clearly unable to handle the step-change of sludge load. This resulted in dramatic temperature change on hearth #3.

Hearth temperature changes were observed to occur only for position changes of the shaft cooling air damper in the 20% to 50% range.

d) Problem Areas

The test results indicate that there is considerable interaction between hearths #5 and #6. It appears that this is due in part to the location of the hearth #5 and #6 thermocouples and the incinerator design. As can be seen in Figure 16, thermocouple #6 is located directly below a drop-hole in hearth #5. This allows radiation from hearth #5 to heat the thermocouple in hearth #6. Consequently, when the hearth #5 burners are increased, both hearth #5 and #6 temperature signals are increased. In a similar fashion, the thermocouple in hearth #5 is located such that it responds to heat from hearth #6 which is convected up through the drop-holes. The heat is then trapped beneath hearth #4 and warms the hearth #5 thermocouple. In order to reduce the interaction between hearth #5 and #6, a plate is being considered that would cover the drop-hole above the hearth #6 thermocouple.

As stated earlier, the hearth process gain was found to be variable and dependent upon the position of the respective hearth burner

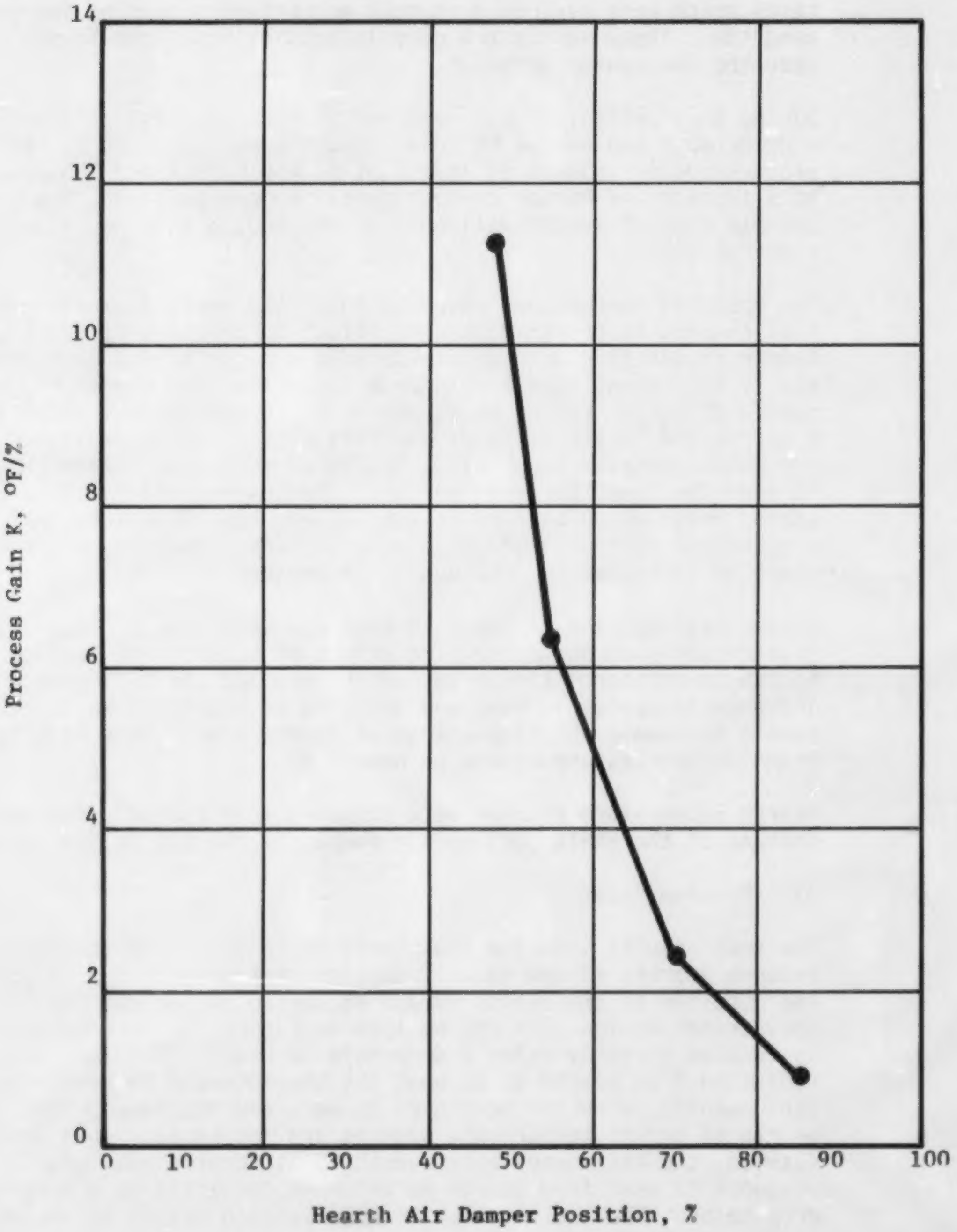


FIGURE 14. PROCESS GAIN OF HEARTH TEMPERATURE RESPONSE TO HEARTH COMBUSTION AIR DAMPER POSITION

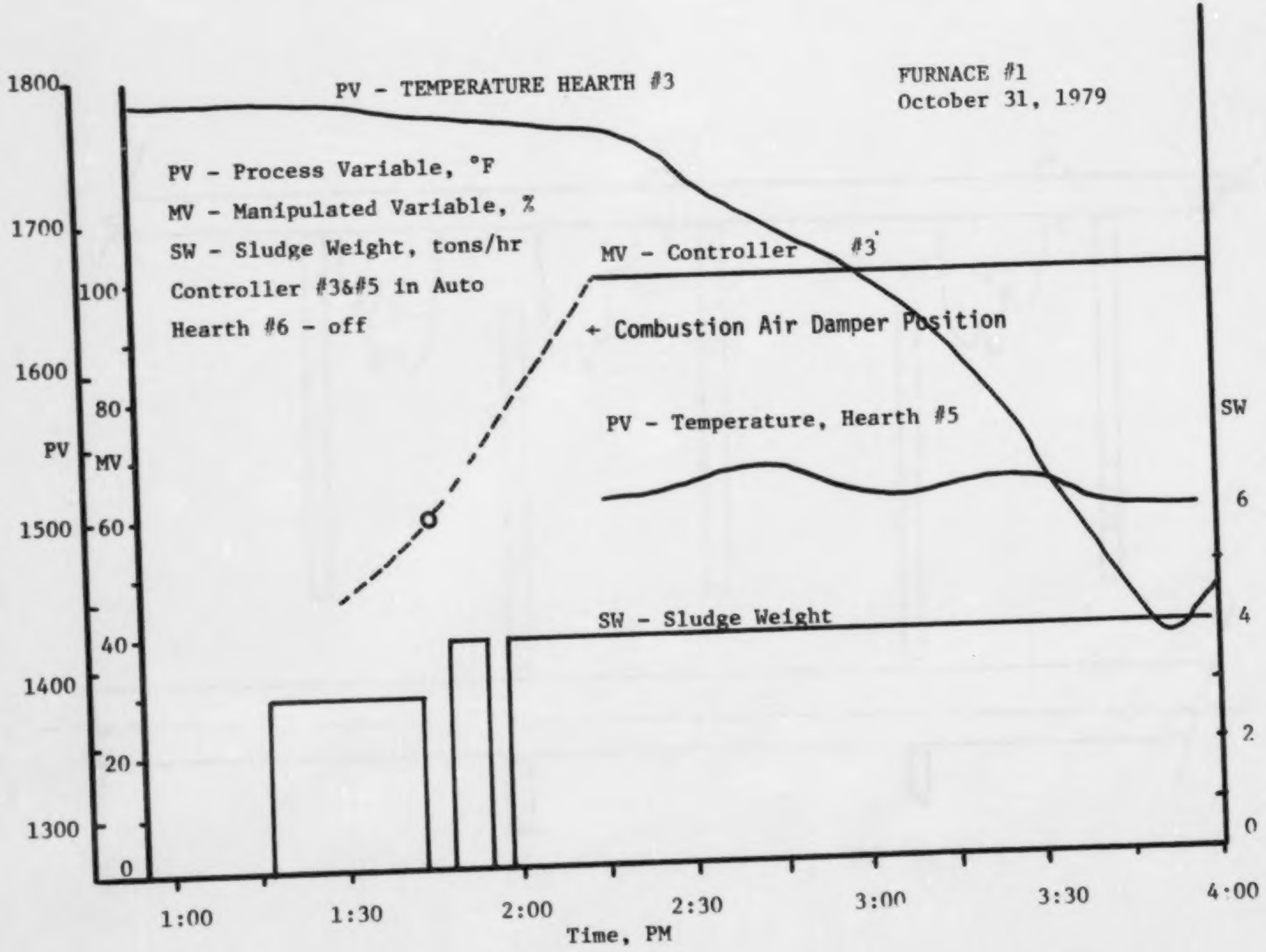


FIGURE 15. EFFECT OF STEP CHANGE IN SLUDGE FEED ON HEARTH TEMPERATURES

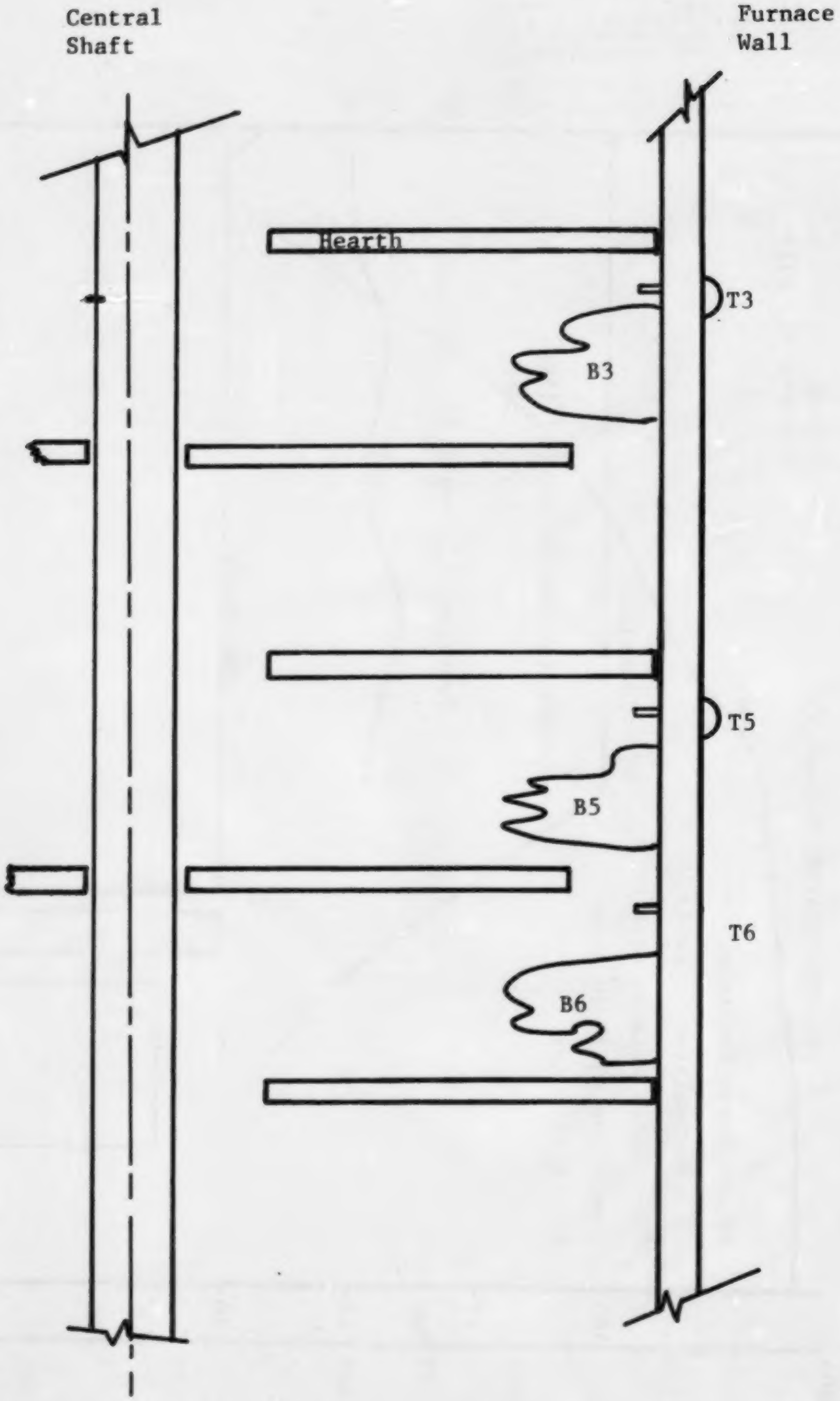

Central Shaft
Furnace Wall
Hearth
T3
B3
T5
B5
T6
B6

FIGURE 16. THERMOCOUPLE POSITION IN INCINERATOR

combustion air damper (Figure 14). This will be compensated by the process control system.

2. Air Flow Characteristics

a) Objective

As with the temperature control system, developments of a control system for air flow will be based upon a model of the air flow dynamics. To determine those dynamics, it is necessary to obtain the relationship between scrubber differential pressure, incinerator pressure, ID fan damper position and scrubber ambient air damper position.

The dynamics of the percent oxygen in the exit gas as it relates to the shaft cooling air damper position, furnace atmospheric air damper position, and the furnace pressure were also tested. Percent oxygen is of considerable importance as it has a significant impact on the amount of fuel consumed by the furnace.

b) Test Set-up

All air flow tests were done on incinerator #1 which is the incinerator to be controlled. For determination of factors affecting oxygen in the incinerator exit gas, the following parameters were monitored on a 3-pen recorder; hearth #1 temperature, sludge feed weight and percent oxygen. The furnace pressure, and the hearth #3, and hearth #5 temperature controllers were maintained in automatic mode. The oxygen analyzer available at the time of testing was a portable unit not intended for continuous measurement. However, the unit can operate continuously for a period of 5 minutes, which was sufficient to measure the exit gas oxygen content resulting from a process change.

To determine the relationship between furnace pressure and scrubber differential pressure, these and the ID fan damper position were recorded on the 3-pen recorder. Step changes of position were then made to the ID fan damper and ambient air damper. The furnace pressure and scrubber differential pressure controllers were in manual mode.

c) Observations

With regard to oxygen content in the exit gas, no change was observed as a result of changes in the atmospheric air damper position. This is due to the relatively small air flow controlled by this damper compared to the total air flow through the incinerator from uncontrolled sources.

As can be seen from Figure 17, changes in the shaft cooling air damper position had no effect on O_2 level at positions below 20% and above 50%. Between 20% and 50%, a total change of 1.5% oxygen

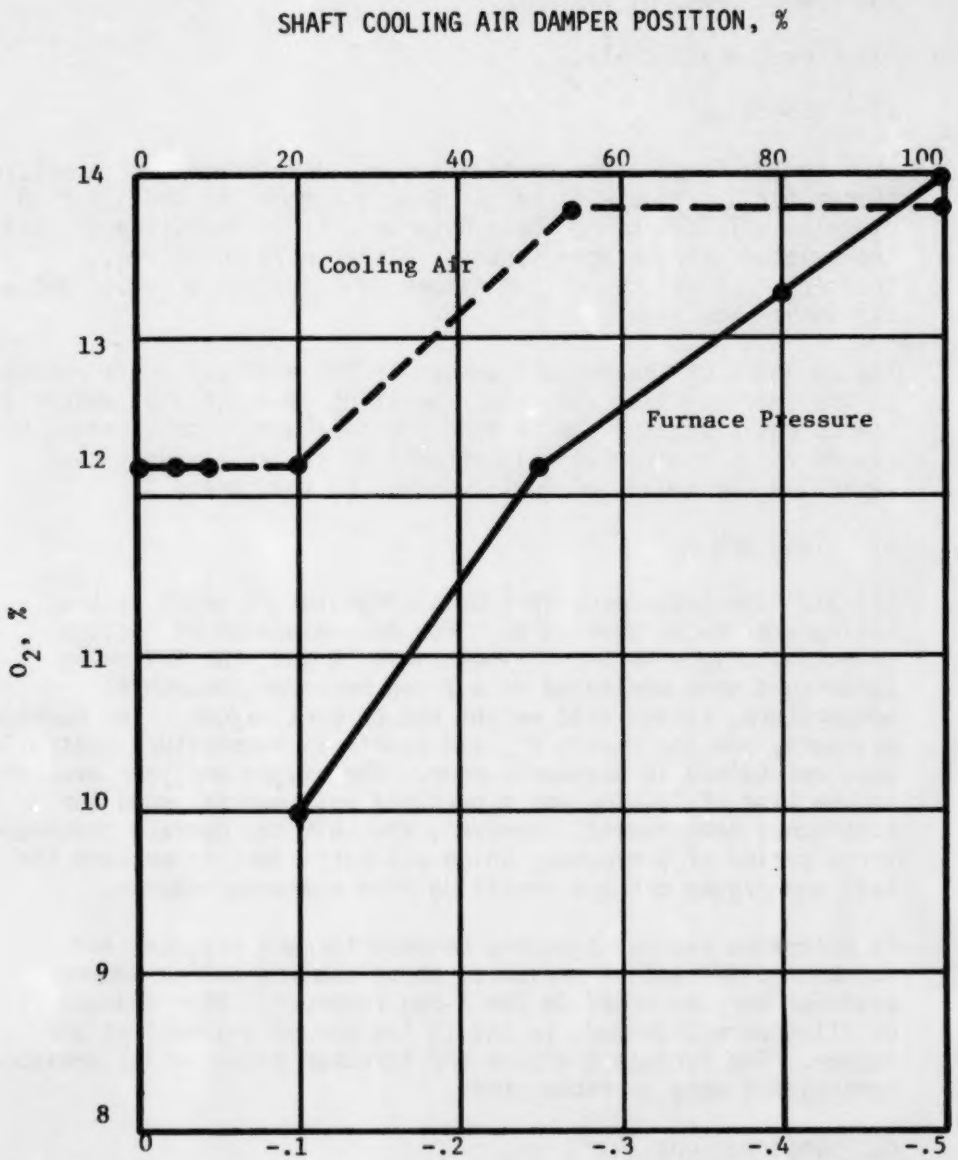


FIGURE 17. EFFECT OF FURNACE PRESSURE & SHAFT COOLING AIR DAMPER POSITION ON OXYGEN FURNACE GAS

was observed.

The strongest influence on percent oxygen was produced by changes in the incinerator draft pressure. This is because the furnace pressure determines the amount of air that enters from the uncontrolled source.

With regard to the relationship involving furnace pressure and scrubber differential pressure, it was found that changes in ID fan damper positions produced rapid changes in both the furnace pressure and scrubber pressure. Conversely, the scrubber ambient air damper had only a minimal effect on the scrubber differential pressure and a slightly greater effect on the furnace pressure.

d) Problem Areas

To enhance the controllability of the excess oxygen and thus reduce excess fuel consumption, reduction of the various air leaks into the furnace is necessary. The major uncontrolled sources are the vacuum filter vent line, the ash slurry tank, the ash ball chute, and the sludge flapper valve on the top of the furnace. Only the vacuum filter vent line is truly uncontrollable. The remaining three items may be modified to reduce the air which passes through, and suggestions have been made to plant management to effect these changes.

The weak response to changes in the scrubber ambient air damper may be attributed to plugging of the ambient air duct. This is being investigated and may result in re-testing of this parameter should the duct be plugged.

D. Process Model

1. Temperature Model

a) Physical Characteristics

The general physical layout of the incinerator is shown in Figure 18. There are three basic zones within the furnace. The top zone (hearths #1 and #2) is for drying of the sludge. This is followed by the burning zone (hearths #3, #4, and #5) in which combustion of the sludge takes place. The last zone is that of final combustion and cooling prior to discharge to the ash hopper. The major disturbance variable to the incinerator temperature is the sludge feed.

Sludge feed to the incinerator is measured by weight scale on the inclined conveyor. As mentioned earlier, the weight signal is very noisy. There is also a long time delay between a change in sludge weight and the observed effect on temperature. The noisy signal and the time delay results in a significant potential for inaccuracy in any feed forward control. This eliminates the use

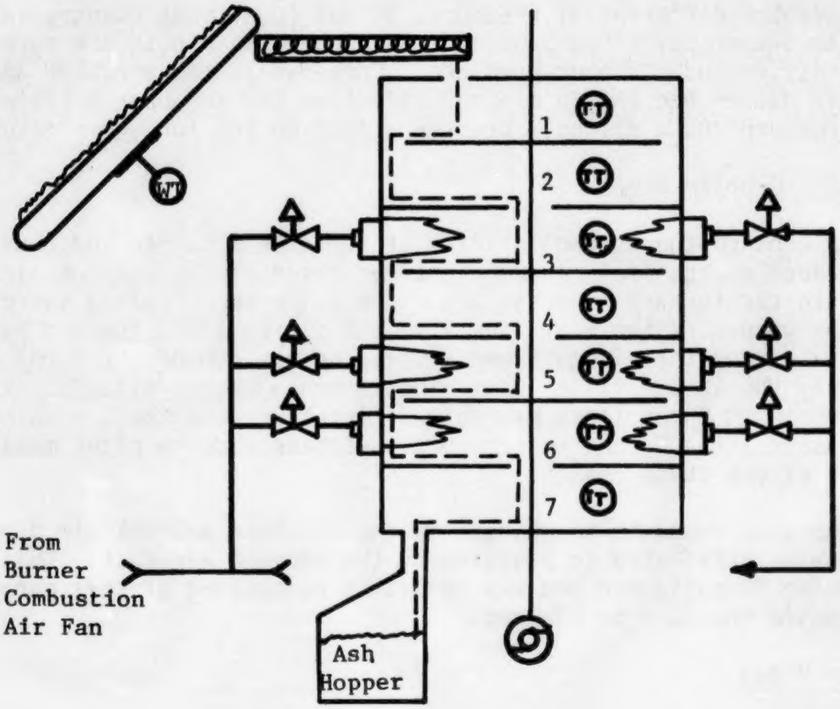

WT
1
2
3
4
5
6
7
TT
From Burner Combustion Air Fan
Ash Hopper

FIGURE 18. SCHEMATIC OF INCINERATOR

of weight as a control parameter. However it will be needed for efficiency calculations and it is anticipated that some signal filtering will make it useful for that purpose. In addition it will be used to prevent gross overload of the incinerator as was illustrated in Figure 15. Observations indicate rabble arm speed changes appear to have a very slow effect on the incinerator temperature. However, variation of rabble arm speed affects the sludge resident time on each hearth and, as such, may play a part in optimization of the incinerator.

In developing the temperature control model, both the sludge feed rate and the rabble arm speed were not incorporated. In both instances it is felt that hearth combustion control, using natural gas or fuel etc., will react quickly to sludge loading changes and will not require the use of these variables, unless an overload of wet sludge occurs.

b) Dynamic Characteristics

A summary of the test results in the multivariable Laplace Transfer model shown in Figure 19. As can be seen from the structure of the mathematical model, there is considerable interaction between hearths #5 and #6. It appears that this is due to the location of the hearth #5 and #6 thermocouples and the incinerator design. It is anticipated that when the drop-hole over thermocouple #6 is physically blocked, the model may then be simplified because the interaction between hearth #5 combustion to the hearth #6 thermocouple will be negligible. This eliminates one interaction and simplifies the control strategy. As stated earlier, the process gain was found to change considerably during the testing and must be treated as a function of combustion air damper position. Thus it is shown as a variable K rather than a constant in Figure 19.

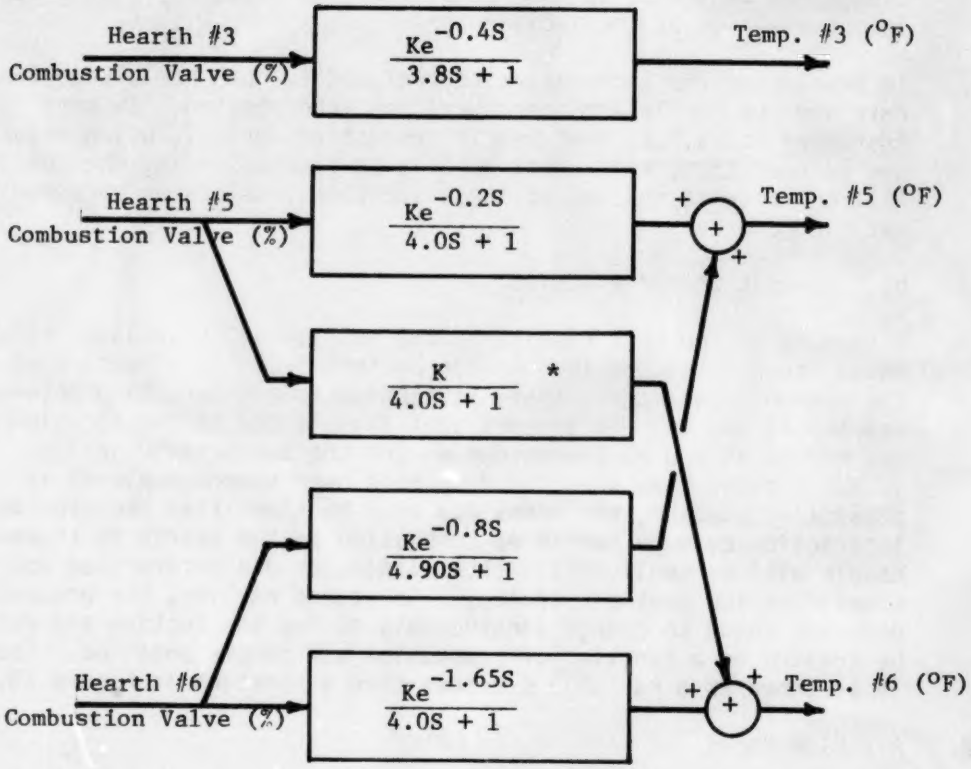
2. Air Flow Model

a) Physical Characteristics

The flow path of gas through the incinerator is shown in Figure 20.

There are three sources of air into the incinerator that can be manipulated by valves. They are burner combustion air, rabble arm shaft cooling air and atmosphere air. The major uncontrolled sources are the vacuum filter vent line, the ash slurry tank, the ash ball chute, and the sludge flapper valve on the top of the furnace.

The ID fan pulls the flue gases out of the furnace and through the scrubber. The flow of gas depends upon five variables. They are, the ID fan damper position, the burner combustion air valve position, the cooling air valve position, the atmospheric air valve position and the ambient air valve position. A change in any one variable will effect the gas flow, resulting in a change in pressure and oxygen. However, the burner combustion air valve


Hearth #3
Combustion Valve (%)
$\frac{Ke^{-0.4S}}{3.8S + 1}$
Temp. #3 (°F)
Hearth #5
Combustion Valve (%)
$\frac{Ke^{-0.2S}}{4.0S + 1}$
+
+
+
Temp. #5 (°F)
$\frac{K}{4.0S + 1}$
*
$\frac{Ke^{-0.8S}}{4.90S + 1}$
Hearth #6
Combustion Valve (%)
$\frac{Ke^{-1.65S}}{4.0S + 1}$
+
+
+
Temp. #6 (°F)

Note

Time constants and dead times are in minutes

FIGURE 19. DYNAMIC TEMPERATURE MODEL OF INCINERATOR

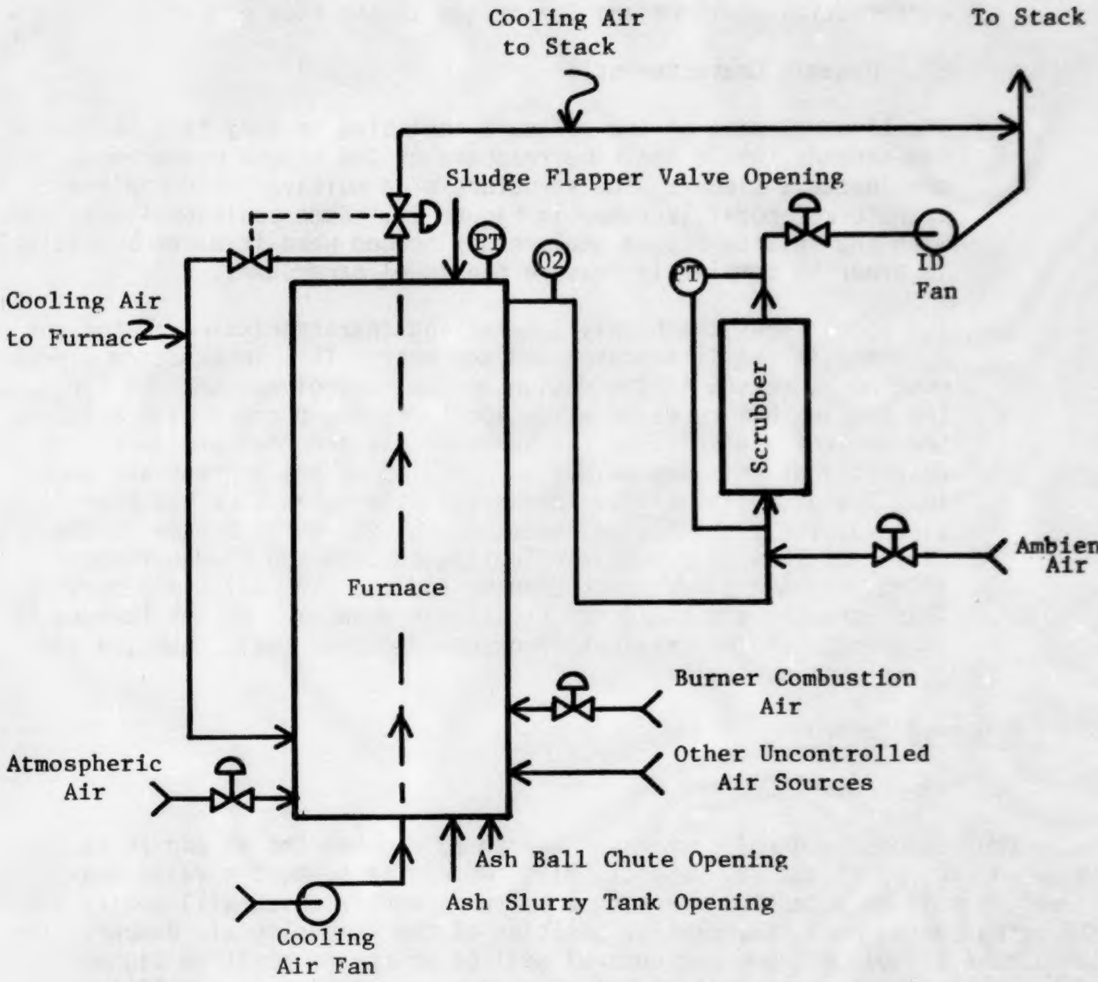

Cooling Air
to Stack
To Stack
Sludge Flapper Valve Opening
PT
O2
PT
ID
Fan
Cooling Air
to Furnace
Scrubber
Ambient
Air
Furnace
Burner Combustion
Air
Atmospheric
Air
Other Uncontrolled
Air Sources
Ash Ball Chute Opening
Ash Slurry Tank Opening
Cooling
Air Fan

FIGURE 20. FLOW PATH OF GAS THROUGH INCINERATOR

is not an independent manipulated variable because it is committed to the control of hearth temperature. In addition, transient tests revealed that the atmospheric air valve position had a negligible effect on gas flow. The three remaining manipulated variables are the inputs to the process control model. The outputs of the model are the furnace pressure, the scrubber differential pressure and the oxygen in the flue gas.

b). Dynamic Characteristic

The time response of the pressure variables is very fast, within a few seconds, while the time response of the oxygen measurement is considerably slower. The structure of a multivariable Laplace transformer model is shown in Figure 21. Some additional testing with the in situ oxygen analyzer is needed when it becomes available in order to completely measure the model parameters.

The model shows the highly interacting characteristics of the gas dynamics in the incinerator and scrubber. This interacting aspect must be addressed in the design of the control system. So far, the testing has revealed an unexpectedly low process gain relating the ambient air valve to the scrubber differential pressure. It appears that this may be due to plugging of the ambient air duct. This process gain will be remeasured after this duct has been cleaned. The testing also revealed high values of oxygen in the flue gas. This is due to air leaking through the sludge flapper valve, the ash slurry tank opening and the ash ball chute opening. This creates a significant excess air problem. If the furnace is to operate at lower values of oxygen and save fuel, then the air leaks must be sealed.

E. Proposed Control

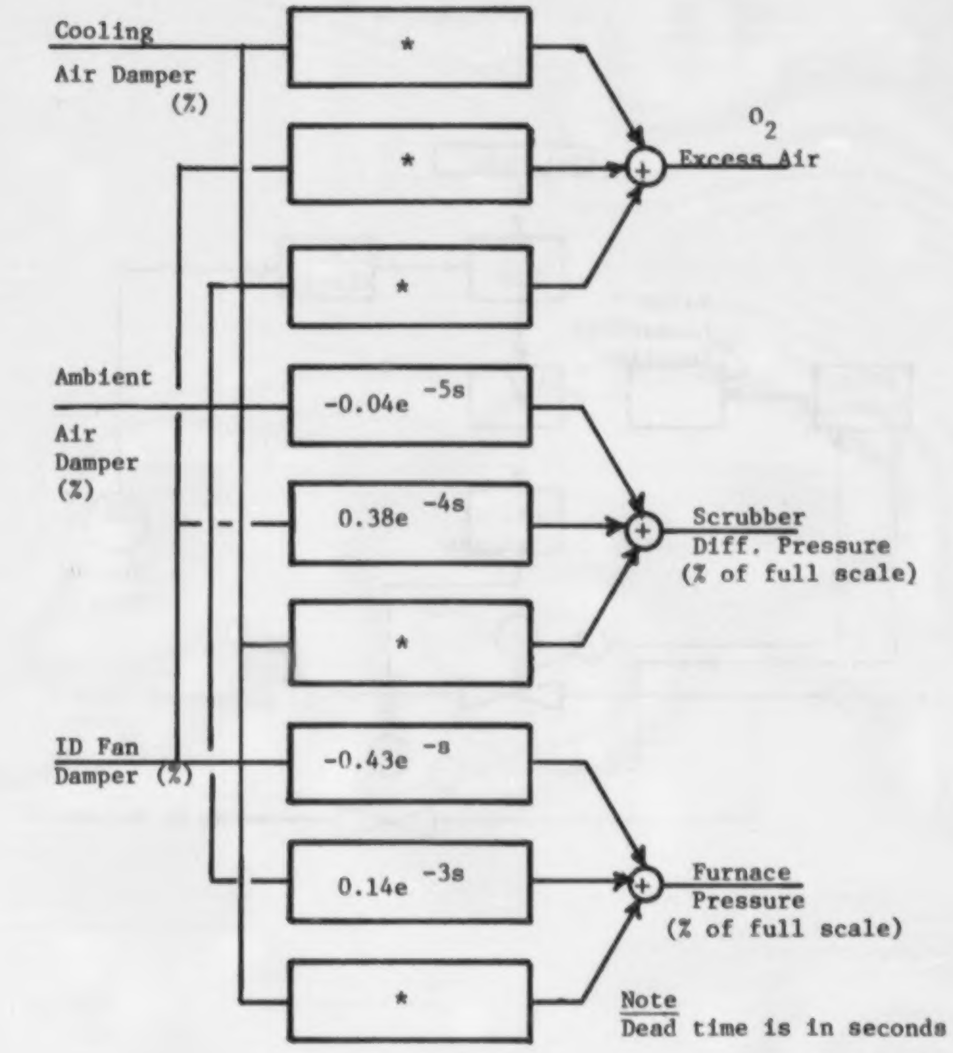
1. Temperature Control

The proposed control system for hearth #3 and hearths #5 and #6 is shown in Figures 22 and 23, respectively. As can be seen, the value non-linearity will be accounted for with a software module which will modify the PID output based upon the relative position of the combusion air damper. The combustion air damper used for control will be whichever position signal is higher. The two burners on each hearth will be controlled in parallel.

Figure 23 outlines hearth #5 and #6 burner control as it is anticipated to exist after the patch of the hearth #5 drop-hole. It is felt that the lead/lag module shown will be sufficient to account for whatever interaction remains between hearth #5 and #6.

2. Air Flow Control

The control system for Air Flow has not been completed yet because as indicated above necessary modeling parameters are not yet available.



Cooling
Air Damper
(%)
Ambient
Air
Damper
(%)
ID Fan
Damper (%)
*
*
*
-0.04e -5s
0.38e -4s
*
-0.43e -s
0.14e -3s
*
O_2
Excess Air
Scrubber
Diff. Pressure
(% of full scale)
Furnace
Pressure
(% of full scale)
Note
Dead time is in seconds

* dynamic model yet to be determined

FIGURE 21. DYNAMIC AIR FLOW MODEL FOR INCINERATOR

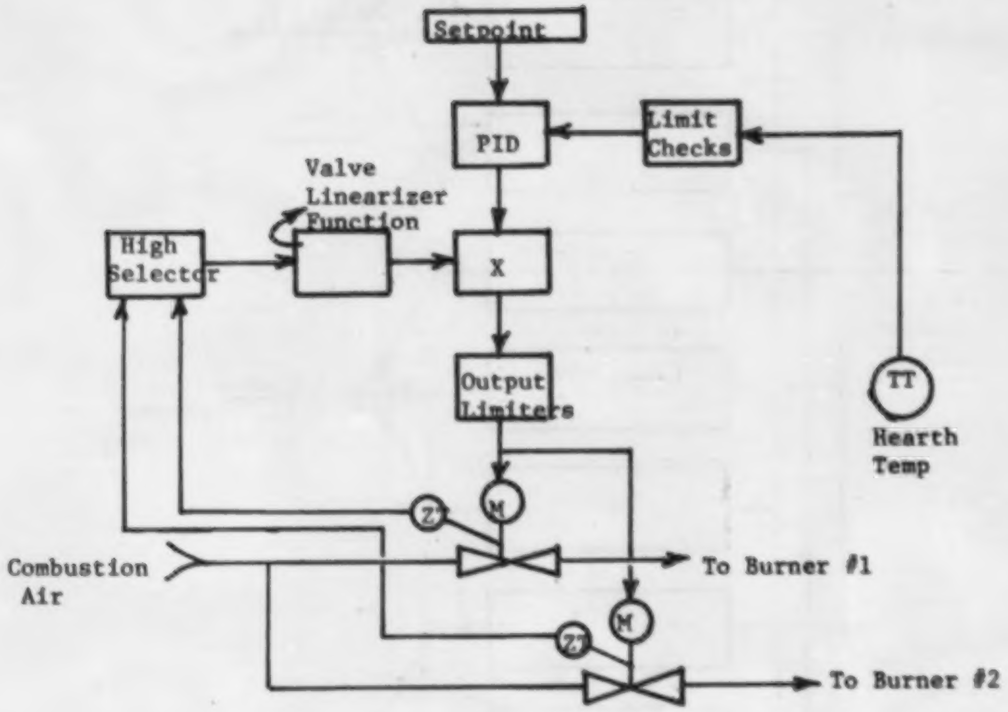

Setpoint
PID
Limit Checks
Valve Linearizer Function
High Selector
X
Output Limiters
TT
Hearth Temp
ZT
M
Combustion Air
To Burner #1
ZT
M
To Burner #2

FIGURE 22. CONTROL SYSTEM HEARTH #3

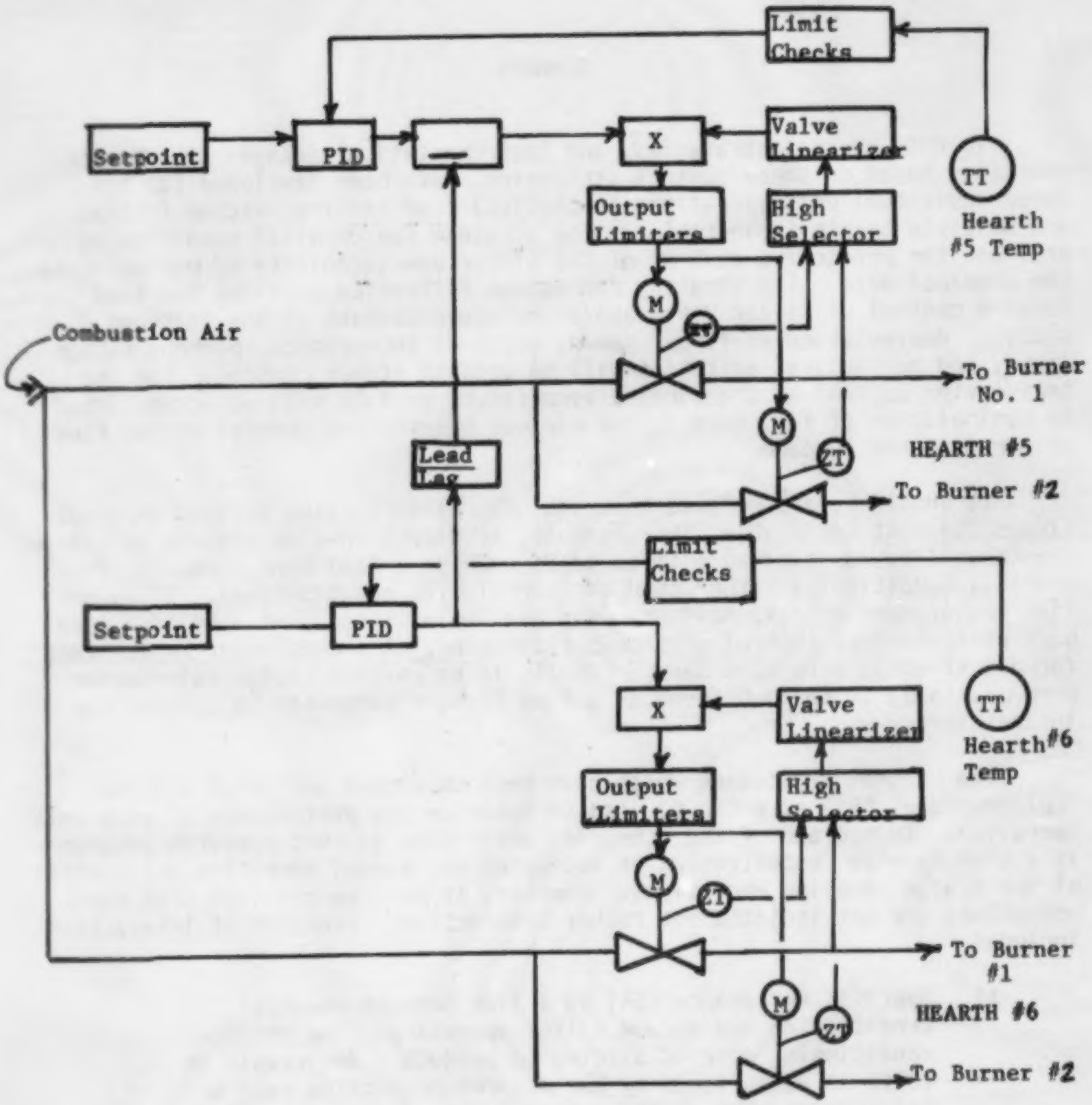

Limit Checks
Setpoint
PID
—
X
Valve Linearizer
TT
Output Limiters
High Selector
Hearth #5 Temp
M
ZT
Combustion Air
To Burner No. 1
M
ZT
Lead Lag
HEARTH #5
To Burner #2
Limit Checks
Setpoint
PID
TT
X
Valve Linearizer
Hearth #6 Temp
Output Limiters
High Selector
M
ZT
To Burner #1
M
ZT
HEARTH #6
To Burner #2

FIGURE 23. CONTROL SYSTEM HEARTHS #5 & #6

SUMMARY

To date control strategies, and instrumentation packages to automate operation based on these control strategies, have been developed for the three individual unit operations of chemical conditioning, vacuum filtration and multiple hearth incineration. The strategy for chemical conditioning provides for predictive control of the sludge dewaterability by manipulating the chemical dose. The strategy for vacuum filtration provides for feed forward control of filter yield and/or moisture content of the filtered sludge. Manipulation of filter speed, depth of submergence, perhaps vacuum level, and perhaps vat agitation will be used to effect control. For the incinerator control of the furnace temperature profile will be accomplished by manipulation of fuel feed to the various hearths and control of the flow of air into the furnace.

It should be appreciated that the control strategies devised were not always the most desireable. For example, if direct on-line measure of sludge dewaterability was available it could be used as a feed back parameter for chemical conditioning rather than the correlation equation used. If an on-line measurement of cake mositure were available it could be used as a feed back parameter for control of vacuum filtration; as a complement to the feed forward strategy developed here. Finally if an on-line sludge calorimeter were available it could be used as a feed forward parameter to control the incinerator.

The control strategies which have been developed and which will be implemented at this site can be used to optimize the performance of each unit operation. Indeed one of the scheduled activities in this research program is a side-by-side demonstration of automated vs. manual operation and control of the sludge handling operations. However, it must be realized that these operations are not isolated but rather interactive. Examples of interaction include:

1) Specific Resistance (SR) is a link between chemical conditioning and vacuum filter operation. The chemical conditioning dose is altered to produce a desireable SR value is established by the desired production rate or moisture content of the vacuum filter discharge. SR of course is not the only parameter which is available to affect vacuum filter performance. Speed of rotation, depth of submergence, etc., can also be used to affect output; but SR is a prime factor and an interactive one as well.

2) Cake production and moisture content are links between the incinerator and the vacuum filter. Production of excessive quantities of wet cake will overload the incinerator, while production of excessive quantities of dry cake may result in excessive temperature build-up and incinerator damage. Production of a cake with thermal characteristics that continually vary is also not good as it can lead to incinerator damage through thermal shock.

It is clear than that optimization of each individual operation will not yield optimum system performance. Rather optimization will require that the interactions noted above be taken into consideration in an overall optimization model. At the present time such a model is being formulated. It will be programmed into the process control computer system which was installed at this site. This computer system has a two fold purpose. It has been used as a data gathering and modeling unit during the time the individual control strategies were under development, and will conduct the automated process control of the treatment line. An interesting feature of this system is the ability to take data as generated and use it to continually update process models. Thus even after the system goes on-line the process models will be automatically updated.

With respect to system optimization the computer based model will use existing input and output goals, process models, and cost information to operate the system on the most economical basis. For example, the program will decide at any instant whether in order to meet temperature goals in the incineration step it is most economical to alter chemical dose, change speed of operation of the vacuum filter, or use more fuel. It would be impossible for a human operator to make such decisions many times a day; because of all the variables involved. Another type of problem the optimization model will address is how to handle maintenance scheduling, temporary changes in system loading or a partial breakdown of equipment. For example, scheduled maintenance and downtime could be handled by running production prior to shutdown at such a rate that ample sludge storage space was available for the time the system was out of service. Eventually both sludge handling lines will be automated which will provide the computer with the ability to shift loading from one line to another.

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TABLE 1A. SLUDGE CONDITIONING DATA SUMMARY

Nov. 1978-Feb. 1979

SSR	PHR	PHF	PHL	ORPR	ORPF	ORPL	T12	T30	T60	CSTR	CSTF	CSTL	SRL	SSL	TL
4.8	6.05	5.70	11.90	-160.	-20.	-325.	6.8	11.0	19.6	343.	45.5	12.1	18.0	4.5	22.0
4.8	-0	-0	12.20	0	-0	-340.	5.7	9.2	13.7	-0	-0	9.7	5.6	4.4	24.5
3.5	6.30	5.25	12.30	-125.	25.	-370.	4.1	7.0	12.2	380.	25.6	8.4	2.9	4.3	25.0
3.5	6.30	5.55	12.10	-125.	-25.	-350.	4.3	7.9	12.5	338.	29.8	8.9	5.5	3.6	24.0
3.8	6.30	5.65	12.30	-160.	-20.	-425.	3.7	5.7	8.7	471.	45.3	10.4	3.3	4.6	22.5
3.8	6.25	5.80	12.35	-150.	-35.	-430.	9.2	10.1	15.3	500.	66.0	9.3	3.3	4.6	25.5
-0	-0	5.85	12.40	0	-65.	-415.	5.7	8.1	14.1	-0	76.8	11.3	4.8	4.9	-0
-0	-0	5.80	12.35	0	-60.	-430.	4.4	7.5	11.7	-0	73.9	11.4	4.7	4.5	-0
5.0	5.75	5.70	12.30	-160.	-110.	-450.	8.3	10.8	15.3	564.	137.0	13.4	4.5	6.3	24.0
-0	-0	-0	12.20	0	-0	-425.	13.2	17.8	26.2	-0	-0	17.2	10.0	5.4	24.0
6.0	6.05	5.70	12.30	-130.	-90.	-415.	14.0	18.1	23.4	451.	199.0	13.2	4.0	6.2	-0
-0	-0	5.60	12.30	0	-105.	-425	21.3	23.5	26.8	-0	93.0	11.7	3.6	6.1	24.0
5.1	5.95	5.80	12.30	-120.	-90.	-420.	12.5	15.0	23.3	492.	191.0	12.5	2.9	6.4	24.0
-0	-0	5.70	12.30	0	-70.	-440.	10.3	13.5	18.9	-0	134.0	12.6	4.1	5.8	-0
-0	-0	-0	12.25	0	-0	-460.	10.1	14.9	23.2	-0	-0	13.3	5.2	5.3	-0
5.5	6.15	6.00	12.35	-155.	-140.	-400.	4.8	7.3	11.3	499.	433.0	12.8	1.5	7.1	25.0
-0	-0	5.40	12.35	0	-35.	-415.	7.0	10.0	15.8	-0	47.2	13.4	4.3	5.2	-0
4.7	5.90	5.65	12.40	-125.	-65.	-395.	7.8	11.4	16.7	440.	119.0	11.6	5.1	5.4	21.5
5.8	6.20	6.25	12.35	-150.	-155.	-410.	6.3	10.4	17.1	317.	122.0	14.8	7.7	5.4	25.0
4.8	6.00	5.65	12.25	-130.	-80.	-450.	8.7	14.0	16.9	325.	101.0	12.0	2.1	5.6	22.0
5.1	5.90	5.80	12.30	-110.	-85.	-440.	12.7	11.3	18.4	452.	121.0	13.3	5.6	5.3	21.0
5.4	6.00	5.75	12.35	-140.	-125.	-420.	4.7	8.0	11.7	607.	275.0	12.4	3.0	5.4	21.0

(continued)

TABLE 1A. (continued)

SSR	PHR	PHF	PHL	ORPR	ORPF	ORPL	T12	T30	T60	CSTR	CSTF	CSTL	SRL	SSL	TL
-0	-0	-0	12.20	0		-370.	6.0	9.6	15.0			14.7	10.0	4.7	19.5
3.9	6.10	5.50	12.30	-105.	-100.	-438.	8.7	10.7	16.3	337.	30.6	10.5	2.5	4.4	23.0
4.2	6.25	5.75	12.30	-150.	-100.	-440.	15.0	17.4	23.0	440.	65.7	10.7	3.4	4.9	20.0
4.3	6.30	5.00	12.30	-140.	0	-514.	8.8	12.6	19.4	522.	27.3	8.9	2.1	5.8	23.0
4.5	6.20	5.75	12.45	-145.	-85.	-472.	11.3	14.0	20.6	314.	50.2	11.6	3.0	5.8	25.0
5.1	6.50	2.70	7.50	-190.	445.	-250.	8.0	11.9	23.6	655.	51.2	14.0	14.0	5.9	23.0
-0	-0	-0	11.65	0	-0	-430.	-0	15.5	22.6	-0	-0	11.6	2.0	5.2	23.0
6.8	6.55	6.00	11.95	-180.	-120.	-460.	8.8	20.3	31.3	656.	102.0	9.2	2.7	5.7	20.5
4.6	6.15	5.30	11.50	-150.	95.	-105.	13.5	16.4	24.9	342.	30.8	8.5	3.0	5.0	18.0
4.9	6.25	5.90	12.05	-135.	-100.	-245.	24.4	24.9	29.6	370.	111.0	8.5	2.6	5.4	20.0
5.3	6.30	6.00	11.55	-165.	-135.	-376.	23.4	35.5	50.7	346.	88.4	9.1	2.8	6.1	23.3
4.8	6.55	6.30	12.35	-160.	-120.	-367.	10.5	15.9	22.3	267.	72.2	10.2	2.8	5.6	23.0
4.3	6.05	5.90	12.20	-115.	-110.	-435.	6.3	9.3	15.2	667.	190.0	9.9	4.2	5.1	18.5
4.2	6.00	5.80	12.20	-135.	-95.	-370.	3.7	7.1	12.8	650.	155.0	10.8	-0	5.0	20.0
4.7	6.05	5.90	12.30	-175.	-175.	-449.	9.1	12.9	17.9	324.	156.0	7.6	2.7	5.9	22.5
4.6	6.20	5.95	12.30	-185.	-160.	-500.	10.9	15.0	22.0	237.	115.0	8.4	2.4	6.1	24.0

* - 0 means no value except for ORPR where 0 means no value

TABLE 2A. SUMMARY OF ON-LINE CONDITIONING DATA

Sept.-Dec. 1979

NUM	PF	PL	DAY	SSR	PHR	PHF	PHL	ORPR	ORPF	VISC	CSTF	SRL
01	12.0	56	910	4.2	5.70	4.80	12.20	-340	70	6.7	19.7	1.35
02	8.5	56	910	4.2	5.70	5.10	12.20	-340	5	6.3	30.2	3.25
03	15.0	72	910	4.0	5.70	4.70	12.15	-338	70	5.7	19.1	1.30
04	14.5	34	910	4.2	5.70	4.75	12.15	-338	75	6.9	19.5	1.85
05	15.0	21	914	3.5	5.90	4.70	11.55	-312	150	5.7	13.3	3.20
06	15.5	14	914	3.4	5.90	4.80	9.80	-315	155	12.4	15.8	25.90
07	16.0	26	914	3.7	5.85	4.70	12.15	-315	185	6.3	14.9	1.65
08	11.5	28	914	3.6	5.85	5.50	12.25	-342	75	7.6	17.8	2.05
09	16.0	47	919	3.9	5.95	4.85	12.30	-420	50	6.1	14.3	1.15
10	13.5	50	919	3.8	6.00	5.35	12.30	-420	45	5.7	15.6	1.35
11	11.5	39	919	3.8	6.00	5.45	12.35	-420	35	6.1	15.4	1.50
12	10.0	43	919	3.8	6.00	5.70	12.35	-420	10	6.7	18.3	1.65
13	10.5	38	919	3.9	5.95	5.30	12.35	-420	50	6.2	16.6	1.50
14	10.0	29	919	3.9	5.95	5.45	12.35	-420	25	7.0	14.5	1.95
15	22.0	41	921	3.8	5.80	4.20	12.15	-338	195	6.9	14.9	0.85
16	15.0	30	921	3.7	5.80	5.05	12.30	-338	70	6.3	16.4	1.45
17	18.0	24	921	3.8	5.80	4.75	12.15	-338	100	6.7	14.9	1.30
18	18.0	21	921	3.8	5.80	4.70	11.90	-338	110	6.0	15.3	2.35
19	11.0	20	921	4.0	5.75	5.00	12.00	-338	75	6.8	16.5	2.45
20	15.5	56	925	3.7	5.90	5.30	12.45	-310	30	7.0	16.5	1.45
21	17.0	53	925	3.9	5.90	5.10	12.40	-310	55	7.9	15.9	1.50
22	14.5	46	925	3.8	5.90	5.20	12.50	-308	-12	8.0	18.3	2.30
23	12.0	45	925	3.9	5.90	5.35	12.50	-310	-30	8.4	23.7	2.10
24	18.5	27	926	3.8	5.85	4.80	12.20	-310	80	4.7	17.1	1.65
25	16.0	22	926	3.8	5.85	5.30	12.20	-315	40	6.8	19.4	1.95
26	15.5	17	926	3.8	5.85	5.40	11.95	-321	30	6.8	21.9	3.05
27	10.0	21	1002	3.8	5.90	5.40	12.20	-345	-125	9.1	26.4	2.80
28	10.0	19	1002	3.7	5.90	5.45	12.10	-345	-115	9.1	33.3	3.50
29	6.3	18	1004	4.2	6.10	5.90	12.30	-335	-60	8.1	56.0	5.60
30	7.9	15	1005	4.2	6.00	5.55	11.85	-340	-65	11.4	55.1	11.90
31	7.7	19	1005	4.3	5.95	5.60	12.25.	-340	-80	10.8	58.5	5.70
32	13.1	26	1010	4.7	6.05	5.40	12.25	-348	-45	6.9	20.6	1.50
33	10.7	13	1010	4.6	6.05	5.60	12.20	-348	-40	9.5	34.1	2.85

(continued)

TABLE 2A. (continued)

NUM	PF	PL	DAY	SSR	PHR	PHF	PHL	ORPR	ORPF	VISC	CSTF	SRL
34	8.3	13	1010	4.6	6.05	5.75	12.15	-350	-60	10.0	52.1	6.00
35	13.0	23	1016	4.9	6.10	5.15	12.20	-355	-5	6.2	25.5	1.30
36	12.0	27	1022	4.1	6.75	5.40	12.25	-370	-180	9.9	32.1	2.05
37	14.0	26	1022	4.0	6.70	5.30	12.20	-370	-140	7.1	28.1	1.75
38	11.5	24	1022	4.2	6.65	5.40	12.10	-370	-110	7.2	31.4	1.75
39	11.5	22	1022	4.2	6.65	5.45	12.00	-370	-100	9.1	31.3	2.70
40	8.5	27	1024	3.7	5.80	5.45	12.25	-350	25	7.6	34.0	2.50
41	8.3	21	1024	3.8	5.85	5.40	12.25	-345	-5	7.7	34.8	2.75
42	7.8	18	1024	3.7	5.80	5.55	12.25	-348	-30	9.9	38.9	4.05
43	7.5	16	1024	3.8	5.80	5.60	12.10	-349	-45	9.3	44.9	6.80
44	17.0	50	1026	3.2	6.30	5.10	12.35	-360	-5	5.7	20.1	1.35
45	15.0	18	1026	3.2	6.25	5.15	12.10	-351	5	6.3	23.4	3.95
46	13.5	36	1029	4.0	6.20	5.05	12.20	-355	-20	8.3	26.0	1.85
47	11.5	34	1029	4.0	6.15	5.35	12.20	-352	-50	8.5	27.5	2.50
48	12.0	22	1029	4.0	6.10	5.40	12.15	-342	-65	7.3	30.4	2.75
49	10.5	14	1029	4.1	6.10	5.60	12.10	-345	-100	7.9	40.0	3.30
50	17.4	20	1204	3.8	5.60	4.50	12.20	-340	-100	8.5	28.3	2.35
51	13.7	23	1205	5.4	6.20	6.05	12.20	-345	-118	32.0	36.9	1.25
52	11.7	23	1205	5.4	6.20	6.05	12.25	-345	-125	19.5	39.1	1.65
53	8.4	16	1205	6.4	6.00	5.90	11.90	-345	-105	39.0	48.5	1.50
54	7.6	16	1206	4.4	5.75	5.60	11.80	-325	-85	8.3	64.2	4.50
55	6.6	20	1206	4.4	5.75	5.60	12.40	-325	-92	7.5	63.4	2.60
56	11.3	21	1207	4.2	5.70	5.70	12.10	-340	-158	9.2	51.7	1.35
57	9.8	21	1207	4.2	5.80	5.60	12.30	-340	-160	17.5	54.1	2.05
58	8.9	16	1207	3.8	5.60	5.55	12.25	-340	-165	6.2	25.1	2.65
59	8.5	14	1207	3.8	5.65	5.65	11.50	-340	-152	8.4	40.7	5.40
60	8.6	17	1207	3.8	5.60	5.60	12.30	-340	-150	8.8	51.7	3.90
61	8.3	39	1212	3.3	5.90	5.65	12.40	-305	-55	11.0	21.3	2.60
62	6.7	11	1212	3.8	5.80	5.65	12.05	-305	-55	10.0	28.2	5.60
63	8.1	21	1212	3.7	5.90	5.30	12.15	-305	-35	13.8	34.4	4.10
64	9.2	26	1213	3.7	5.50	4.80	12.10	-295	92	10.9	30.4	2.25
65	7.7	21	1213	3.7	5.50	4.90	12.10	-295	50	12.6	36.1	4.10

IMPACT OF INNOVATIVE AND ALTERNATIVE TECHNOLOGY IN THE UNITED STATES IN THE 1980'S

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I. INTRODUCTION

Recent History of Wastewater Pollution Control in the United States

The design and construction of municipal wastewater treatment facilities in the United States in the 1970s have been significantly impacted by federal laws since 1956 when the Federal Water Pollution Control Act (PL 84-660) was passed. The role of the federal government during this time has been to establish national policies and goals for abatement of pollution and protection of health and the environment and to provide federal support for the construction of necessary treatment works.

The Congress of the United States recognizes and protects the rights of the fifty states as being primarily responsible for controlling water pollution within their boundaries. Between 1956 and 1972, 13,764 municipal treatment plants were constructed with federal assistance totaling 5.2 billion dollars which represented an average of 37% of the total cost of construction.

In 1972 the Federal Water Pollution Control Act Amendments were passed (PL 92-500). This act reaffirmed and expanded the goals of earlier federal legislation and set specific targets to achieve by July 1, 1983 water that is clean enough for swimming, recreational use, protection and propagation of fish, shellfish, and wildlife; and by 1985 to eliminate all discharges of pollutants in the Nation's waters. Two major provisions of this act were to establish a uniform national minimum effluent standard of secondary treatment and to authorize an increase in the federal share of construction to 75% of the total project cost. The total amount of funds authorized from 1972 thru 1980 by the Federal Water Pollution Control Act Amendments (PL 92-500 and PL 95-217) was 33.98 billion dollars (1).

This increase in federal support of municipal construction and the promulgation of a uniform national minimum effluent standard has resulted in accelerated construction of secondary treatment facilities. The following table illustrates the increases in population served and total number of secondary and tertiary municipal treatment plants constructed in the United States from 1968 to 1979.

TABLE 1

INCREASE IN POPULATION AND NUMBER OF MUNICIPAL TREATMENT PLANTS CONSTRUCTED IN THE UNITED STATES FROM 1968 TO 1979 (2)(3)(4)

Type of Treatment	1968 Pop. Served x 10^6	1968 No. Facilities	1979 Pop. Served x 10^6	1979 No. Facilities
Secondary	81	9,333	110	13,108
Tertiary	3(a)	10	5	1,207

(a) Population Estimated

Current Treatment Practice

The design of municipal treatment plants in the United States during the 1960s and early 1970s has followed traditional engineering practices. Planning emphasis has been placed on regional water quality and stream basin analyses necessary to establish levels of point source and non-point source controls that are adequate to protect federally established national water quality goals under PL 92-500. All streams in the United States are classified by the states as to intended use and quality. There are 6,493 total stream segments with 3,299 being classified as water quality limited and 3,194 classified as effluent limited (5). The national minimum secondary effluent standard applies to the effluent limited segments and higher levels of treatment are required by the water quality limited stream segments. For the water quality limited stream segments, waste load allocation studies were undertaken to establish point source effluent limits thus providing a specific design goal for the planning, engineering, and design of wastewater treatment plants. Process selection in the past has largely been based on a cost effectiveness analysis for meeting a particular effluent goal rather than a cost benefit analysis that considers stream impacts. Recently, decisions to construct advanced waste treatment facilities are considering the costs and benefits of improved water quality.

Design practice in the United States for municipal treatment plants from the early 1950s to the early 1970s has been strongly influenced by the state governments because of their responsibility for the review and approval of facility plans and specifications. All but a few of the 50 states have adopted specific design criteria for sizing and selection of components and equipment used in the commonly employed biological treatment processes of activated sludge, trickling filter, and stabilization ponds. The recommended standards for sewage works established by the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers in 1947, now referred to as the Ten State Standards, has been the dominant influence in municipal wastewater treatment plant design practice during this time period.

Standard design requirements such as those included in the Ten State Standards are being increasingly viewed by present-day designers as overly conservative and not widely applicable to the more sophisticated designs. A recent study by USEPA of 29 non-structural factors affecting the construction cost of municipal treatment plants indicated that the use of overly conservative standard design requirements was the most important single cost factor for meeting a specific effluent design goal. Elimination of standard design requirements was estimated to reduce total construction cost by 10-20% (6). Recognizing the increased per-user cost of wastewater treatment, the USEPA, first with enactment of PL 92-500 and again with the 1977 amendments (PL 95-217), has placed strong emphasis on both the consideration of a wide range of treatment alternatives for a given treatment plant and a rigorous cost effectiveness analysis to determine the lowest cost design. This has been vitally important in view of the increasing number of technologies that have become available for treatment of municipal wastewaters in the 1970s.

A draft Innovative and Alternative Technology Assessment Manual published in October of 1979 by EPA-OWPO and ORD summarized the cost, applicability, and energy utilization of 117 separate technologies currently being used for municipal wastewater treatment (7).

New National Goals

Because of the large, sparsely populated land areas of the United States compared to Japan and the increased cost of centralized treatment and disposal by conventional technologies, recent federal legislation including both the FWPCA of 1972 and the 1977 amendments to this law have strongly encouraged the consideration of technologies such as land application that reclaim and recycle wastewater and wastewater constituents, conserve energy, reduce overall cost and reduce the migration of pollutants. Technology-related goals, policies, and key regulations and guidelines of each law are summarized in Table 2.

The basis for an increased federal role in identifying and encouraging specific methods of treatment and technologies is clearly evident from the specific changes in the language of the legislation between the FWPCA of 1972 and the 1977 amendments as summarized in Table 2. The overall thrust of the new law is to more clearly identify technological goals and to offer specific incentives for their use. The incentives to encourage Innovative and Alternative (I/A) Technologies are an increase in federal construction grants from 75% to 85% using special set-aside funds of 2% in fiscal years 1979 and 1980 and 3% in fiscal year 1981 that can only be used to increase the federal grant share of the capital cost at I/A facilities. This increase underscores the intent of Congress to increase the use of Innovative and Alternative Technology to over 25% of all construction grants awarded in fiscal year 1981. This act has effectively set the stage for dramatically changing the practice of sanitary engineering design in the United States over the next ten years.

TABLE 2

THE FEDERAL WATER POLLUTION CONTROL ACT OF 1972 AND THE 1977 AMENDMENTS

THE FEDERAL WATER POLLUTION CONTROL ACT OF 1972	
GOALS AND POLICIES	**MORE IMPORTANT REGULATIONS, REQUIREMENTS, AND GUIDELINES**
Goals: • Eliminate the discharge of pollutants by July 1, 1985 • Achieve interim goal which provides for the protection and propagation of fish, shellfish, and wildlife and provides for recreation in and on the water by July 1, 1983 (fishable, swimmable goal) Policies: • Eliminate the discharge of toxic pollutants • Provide federal assistance for construction of publicly owned treatment works • Development and implementation of areawide planning to control point source and non-point source wastes • Conduct research and demonstration studies to develop technology • Promote efficiency in government management and implementation of program	• Requires basin with planning for point source and non-point source wastes • Requires best practicable waste treatment technology • Pequires technology that: (a) considers advanced waste treatment (b) reclaims and recycles water (c) confines disposal of pollutants (d) recycles pollutants (e) encourages revenue producing facilities (f) combines open space and recreational considerations • Requires infiltration/inflow studies • Provides 75% grants for publicly owned treatment works • Requires user charge and industrial cost recovery • Established national minimum treatment level of secondary treatment
POLICIES, IMPLEMENTING REGULATIONS, AND GUIDELINES ADDED BY THE 1977 AMENDMENTS TO THE ABOVE 1972 ACT (THE 1977 AMENDMENTS ARE ALSO KNOWN AS THE CLEAN WATER ACT OF 1977 (PL 95-217)	
Goals: • None Policies: • Authority of each state to allocate quantities of water within its jurisdiction shall not be superceded or abrogated by this Act • Federal agencies shall cooperate with state and local agencies to develop comprehensive solution to prevent, reduce, and eliminate pollution	**MORE IMPORTANT REGULATIONS, REQUIREMENTS, AND GUIDELINES** • Requires consideration of innovative and alternative technologies for all facility plans initiated after Sept. 30, 1978 • Increases federal support from 75% to 85% for innovative and alternative technologies • Provides guarantee for replacement of innovative and alternative technology plant components that fail to meet design or performance standards • Identifies individual technologies to be encouraged such as land application • Provides a 15% cost preference in the cost effectiveness analysis for innovative and alternative technologies • Extends federal eligibility to privately owned treatment facilities and alternative wastewater collection systems • Allocates 4% of federal construction grant funds for construction of treatment facilities in small communities

The new national goals place strong emphasis on the use of natural systems and improved biological processes that conserve energy and reduce costs and recycle wastewater and wastewater nutrients for beneficial uses.

II. INNOVATIVE AND ALTERNATIVE TECHNOLOGY

Following enactment of the 1977 amendments, the USEPA moved forward swiftly in developing the regulations and guidelines necessary to achieve the goals of the act while still maintaining the momentum of the Construction Grants Program. Table 3 summarizes the ten sections of the law and eight separate regulations that pertain to innovative and alternative technology.

The legislation and regulations noted above formed the basis of the Innovative and Alternative Technology Program now in operation as part of the USEPA's Construction Grants Program. An assessment manual (7), promulgated by USEPA as a part of the 35.908 Innovative and Alternative Technology regulation, defines I/A Technology and forms the basis for classification and screening of all municipal treatment technologies as conventional concepts of treatment, Alternative Technology, or Innovative Technology.

- Innovative Technology is defined as processes and techniques that are developed methods which have not been fully proven under the circumstances of their contemplated use and which represent a significant advancement over the state of the art in terms of meeting the national goals.

- Alternative Technology is defined as wastewater treatment processes and techniques that are proven methods which provide for the reclaiming and reuse of water, productively recycle wastewater constituents, or otherwise eliminate the discharge of pollutants or recover energy.

- Conventional concepts of treatment are generally defined as biological or physical chemical processes with direct point source discharge to surface waters.

The regulations further distinguished Innovative and Alternative Technology by specifically identifying the Alternative Technologies and by establishing six criteria for qualifying any technology as Innovative. The general classification scheme is shown in Figure 1.

Any technology in the Alternative category as defined by the regulations can qualify as Innovative by meeting any one of the six qualifying criteria shown. Any technology that has not been identified as Alternative falls into the category of conventional concepts of treatment and can qualify as innovative by meeting either the cost or energy criteria. These criteria are described as follows:

TABLE 3

SUMMARY OF INNOVATIVE AND ALTERNATIVE TECHNOLOGY LEGISLATION AND REGULATIONS

LEGISLATION PL 95-217, DECEMBER 27, 1977

Section	
201(d)	- Encourages the design and construction of revenue producing facilities
201(g)5	- Requires all applicants to study innovative and alternative technology
201(i)	- Encourages energy conservation in the design of all publicly owned treatment works
201(e)	- Encourages the reduction of total energy requirements in the design of publicly owned treatment facilities
201(j)	- Provides for 15% cost preference in the cost effectiveness analysis for all innovative and alternative technology
202(a)2	- Increases federal grant from 75% to 85%
202(a)4	- Limits grant eligibility to publicly owned treatment works (excludes sewers and sewer rehabilitation)
304(d)3	- Requires EPA to develop guidelines for innovative and alternative technology
205(i)	- Authorizes innovative and alternative funding set-asides for fiscal years 1979, 1980, and 1981

REGULATION 40 CFR 35, SEPTEMBER 27, 1978

Regulations	
35.908	- Describes innovative and alternative policy, funding, priority scheduling and replacement provisions of the Act
35.915(a)(1)	- Describes state priority system
35.915(e)	- Provides for USEPA review of state priority system
35.917-1(d)(8)(9)	- Requires innovative and alternative technology and energy review
35.915(b)	- Provides for establishment of state reserve set asides to increase federal share of cost from 75% to 85%
35.930-5(b)	- Provides for 75% to 85% grant increase for new and replacement innovative and alternative projects
35.935-20	- Provides for EPA post construction evaluation and inspection for five years
35.936-13	- Provides exclusion to non-restrictive specifications for certain innovative and alternative technologies and "buy American" provisions

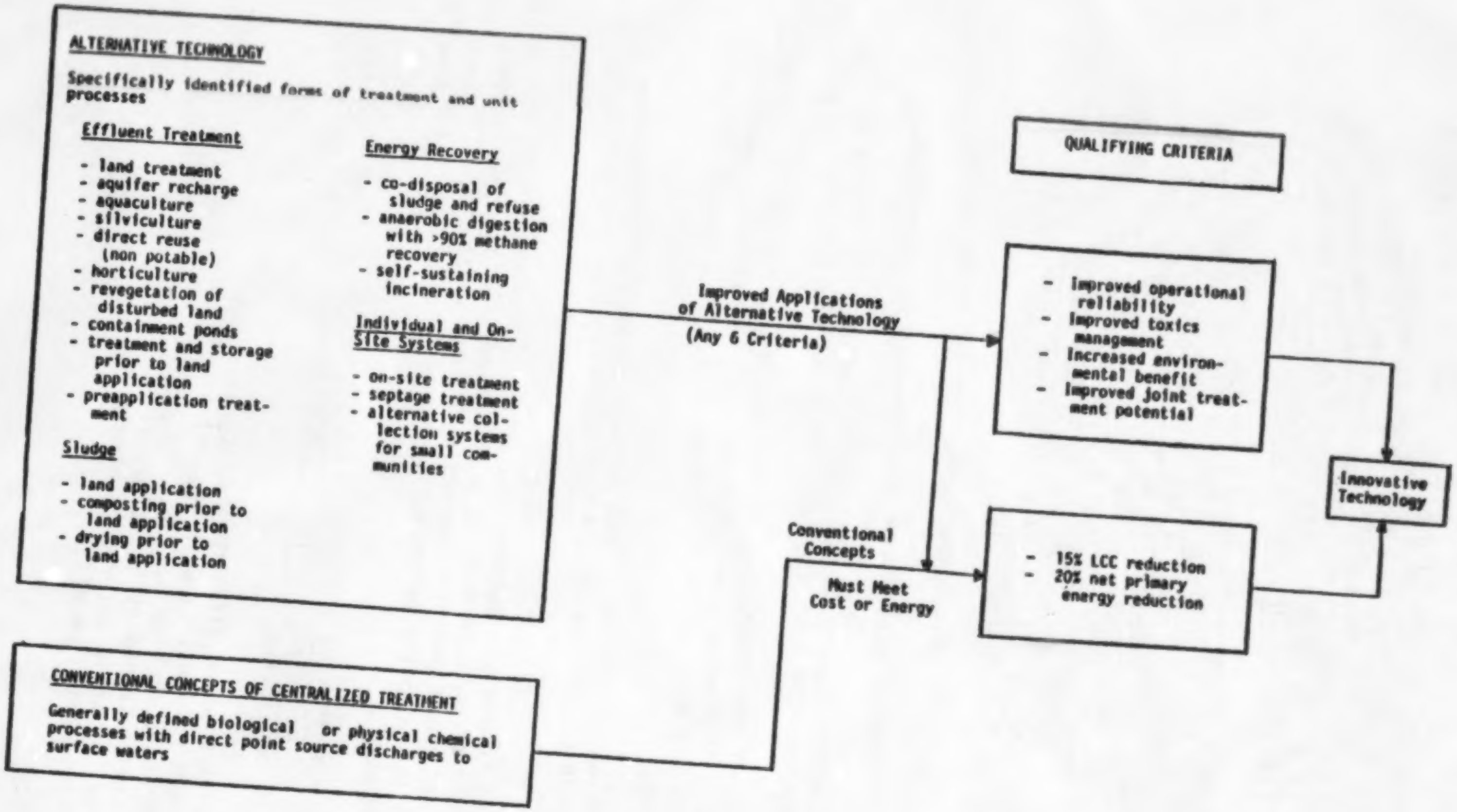

ALTERNATIVE TECHNOLOGY
Specifically identified forms of treatment and unit processes
Effluent Treatment
- land treatment
- aquifer recharge
- aquaculture
- silviculture
- direct reuse (non potable)
- horticulture
- revegetation of disturbed land
- containment ponds
- treatment and storage prior to land application
- preapplication treatment
Sludge
- land application
- composting prior to land application
- drying prior to land application
Energy Recovery
- co-disposal of sludge and refuse
- anaerobic digestion with >90% methane recovery
- self-sustaining incineration
Individual and On-Site Systems
- on-site treatment
- septage treatment
- alternative collection systems for small communities
CONVENTIONAL CONCEPTS OF CENTRALIZED TREATMENT
Generally defined biological or physical chemical processes with direct point source discharges to surface waters
Improved Applications of Alternative Technology
(Any 6 Criteria)
Conventional Concepts
Must Meet Cost or Energy
QUALIFYING CRITERIA
- Improved operational reliability
- Improved toxics management
- Increased environmental benefit
- Improved joint treatment potential
- 15% LCC reduction
- 20% net primary energy reduction
Innovative Technology

FIGURE 1

GENERALIZED CLASSIFICATION OF INNOVATIVE AND ALTERNATIVE TECHNOLOGY

1. The life cycle cost of the eligible portions of the treatment works excluding conventional sewer lines is at least 15% less than that for the most cost effective alternative which does not incorporate innovative wastewater treatment processes and techniques (i.e., is no more than 85% of the life cycle of the most cost effective non-innovative alternative).

2. The net primary energy requirements for the operation and maintenance of the eligible portions of the treatment works excluding conventional sewer lines are at least 20% less than the net energy requirements of the least net energy alternative which does not incorporate innovative wastewater treatment processes and techniques (i.e., the net energy requirements are no more than 80% of those for the least net energy non-innovative alternative must be one of the alternatives selected for analysis under Section 5 of Appendix A.

In the above life cycle cost comparisons, the following apply:

- The non-innovative alternative must be clearly identified.

- The cost effectiveness of the non-innovative alternative will be judged against the best available state of the art cost information.

- The basis of the comparison is the lowest present worth cost with the cost effectiveness analysis for each system being conducted in accordance with USEPA prescribed Cost Effectiveness Analysis Guidelines.

- Aggregation of component cost savings is permitted providing all applicable provisions of the Cost Effectiveness Analysis Guidelines are met.

- The cost comparison between the proposed innovative and non-innovative alternatives must be made on a completed treatment works basis (grant eligible portions excluding conventional sewer lines) even though the proposed potentially-innovative portion is a sub-system or component.

- In the comparative analysis, both systems must provide equivalent levels of pollutant control. Equivalency of the following factors must be considered.

    - Design minimum effluent quality standards
    - System reliability with respect to effluent quality and residual disposal
    - Residual treatment and disposal
    - Level of toxic material control
    - Environmental benefit

The energy analysis compares the net primary energy utilization of the proposed innovative alternative with that of the least net primary energy utilization non-innovative alternative. In this analysis, the required 20% net primary energy savings must be made on the total treatment works basis (eligible portions excluding conventional sewer lines) including treatment and disposal of all residuals except where an upgrading or expansion of an existing treatment works is encountered. In this case, only the treatment works associated with the increased capacity and/or level of treatment shall be considered for inclusion in the energy analysis. All increases in energy use in other portions of the plant must be accounted for in the above net 20% energy savings calculation. The energy audits for the potentially innovative and non-innovative alternative technologies must be made on an equivalent basis. The following general requirements and conditions should be considered.

1. The boundaries and boundary conditions of system or system components receiving energy audits must be fully described including all flow streams entering or leaving the treatment works.

2. A materials balance for all significant influent and effluent streams including flow, mass, temperature, etc., must be included in the analysis.

3. Differences in hydraulic profile required of different treatment systems must be accounted for in the energy analysis.

4. The system's energy balance must include the treatment and disposal (including transportation) of all residuals.

5. In determining the energy consumed to dewater sludge, the energy balance point is the sludge stream entering the dewatering process or the thickening process when thickening is required. All energy balances required to document energy recovery will be based on annual values. The sludge mass to be used in the analysis is the average annual sludge mass projected over the project planning period.

6. For processes where the energy utilization is a function of influent flow, the energy analysis must consider the increase in plant flow over the project planning period as permitted by the USEPA Cost Effectiveness Analysis Guidelines.

7. The net primary energy reduction of 20% does not distinguish the form or location of the energy savings. The savings should be computed at the boundary of the proposed treatment works in BTU or KwH/year. The conversion efficiency of fossil fueled electrical power generation and distribution to the plant site may be taken as 32.5%. A heat to electrical power conversion of 10,500 BTU/KwH may be used in this calculation.

8. Energy saving credits may be granted for lower grade fuel substitutions.

9. Net primary energy is the net energy consumed for the complete treatment of wastewater, including the transportation and ultimate disposal of all residuals.

In addition to meeting cost and energy criteria, innovative technology must have some element of increased risk and associated benefit. Traditional engineering practice has always dictated a very low element of risk for the construction of full scale public works projects supported by federal expenditures. In passing PL 95-217, Congress clearly intended that a higher degree of risk be permitted for innovative technology. The permitted degree of risk should be compared to the potential for significant state-of-the-art advancement. High risk, high potential state of the art advancement projects may be judged acceptable for funding where high risk, low potential state of the art advancement projects may be deemed unacceptable. The concept of risk vs. stage of technology advancement used in this program is shown in Figure 2.

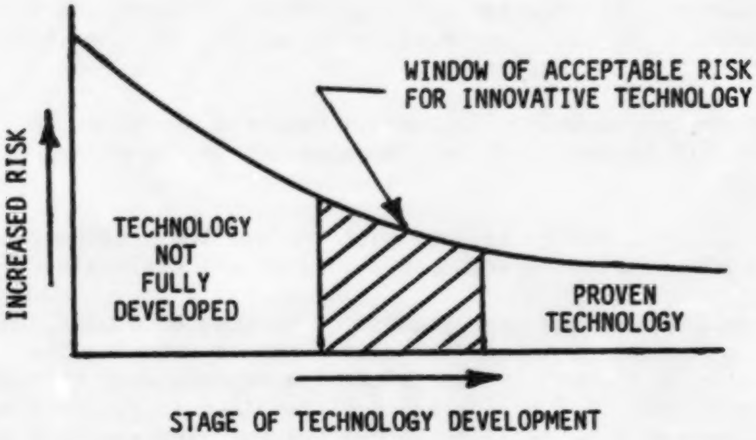

WINDOW OF ACCEPTABLE RISK FOR INNOVATIVE TECHNOLOGY
INCREASED RISK
TECHNOLOGY NOT FULLY DEVELOPED
PROVEN TECHNOLOGY
STAGE OF TECHNOLOGY DEVELOPMENT

FIGURE 2

WINDOW OF ACCEPTABLE RISK

Technologies to the left of the window of acceptable risk in Figure 2 are considered not fully developed and are too risky compared to benefits for full-scale construction. Technologies to the right of the window of risk are considered proven and should not benefit from the increased innovative technology funding provisions. The innovative program is designed to encourage designers to develop technologies that are within the window of acceptable risk. This calls for a conscious decision on the part of the designers to depart from traditional practice and propose higher than normal risk designs that have increased potential for achieving cost, energy, or other benefits. This is a key element in this new national program.

From a procedural point of view, all facility plans for the construction of municipal treatment plants proposed by consulting engineers are evaluated by the state and EPA regional offices in a three-step process shown in Figure 3.

The first step is the determination if the proposed technology involves increased risk (Point A). The second step is the determination if the technology meets the innovative qualifying criteria (Point B). The third and final step is the determination if the proposed technology is the most cost effective system to meet the effluent quality goals. In this last step, all I/A Technologies are given a 15% advantage or preference in the required cost effectiveness analysis.

I/A Technologies passing the three step qualifying process are awarded an 85% federal grant instead of the normal 75% grant. These projects also qualify for a replacement grant if the higher risk technology fails to meet design or performance standards within the first two years of operation.

A critical element of the entire I/A Program is the method by which the states receive and distribute the increased grant funds. Each state is allocated a portion of the federal construction grant budget each year according to assessment of need. Two percent of this allotment for fiscal years 1979 and 1980 and three percent for fiscal year 1981 is set aside to be used only to increase the 75% federal share to 85%. A minimum of one half of one percent must be spent on Innovative rather than Alternative Technology. A strong incentive for the states to encourage the expenditure of their set aside funds is the provision that if they are not spent, the states cannot participate in any reallotment of the 75% funds remaining at the end of a given fiscal year.

III. RESPONSE TO AND SUCCESS OF THE PROGRAM DURING THE FIRST ONE AND ONE-HALF YEARS

The USEPA Construction Grants Innovative and Alternative Technology Program has been in operation for approximately 1 1/2 years. Initial national management efforts during the first year emphasized information dissemination and training of design engineers as well as state and federal review authorities. A comprehensive Innovative and Alternative Technology Assessment Manual was prepared during the first year and distributed to over 6,500 engineers (7). This manual summarizes the federal legislation and regulations, provides detailed procedural guidance for identifying and judging I/A Technologies, and summarizes the cost, energy, and performance of 117 state-of-the-art technologies used as the baseline of conventional technology. Additionally, over 30 separate technical training seminars were held to acquaint over 2,600 designers with the I/A Program requirements.

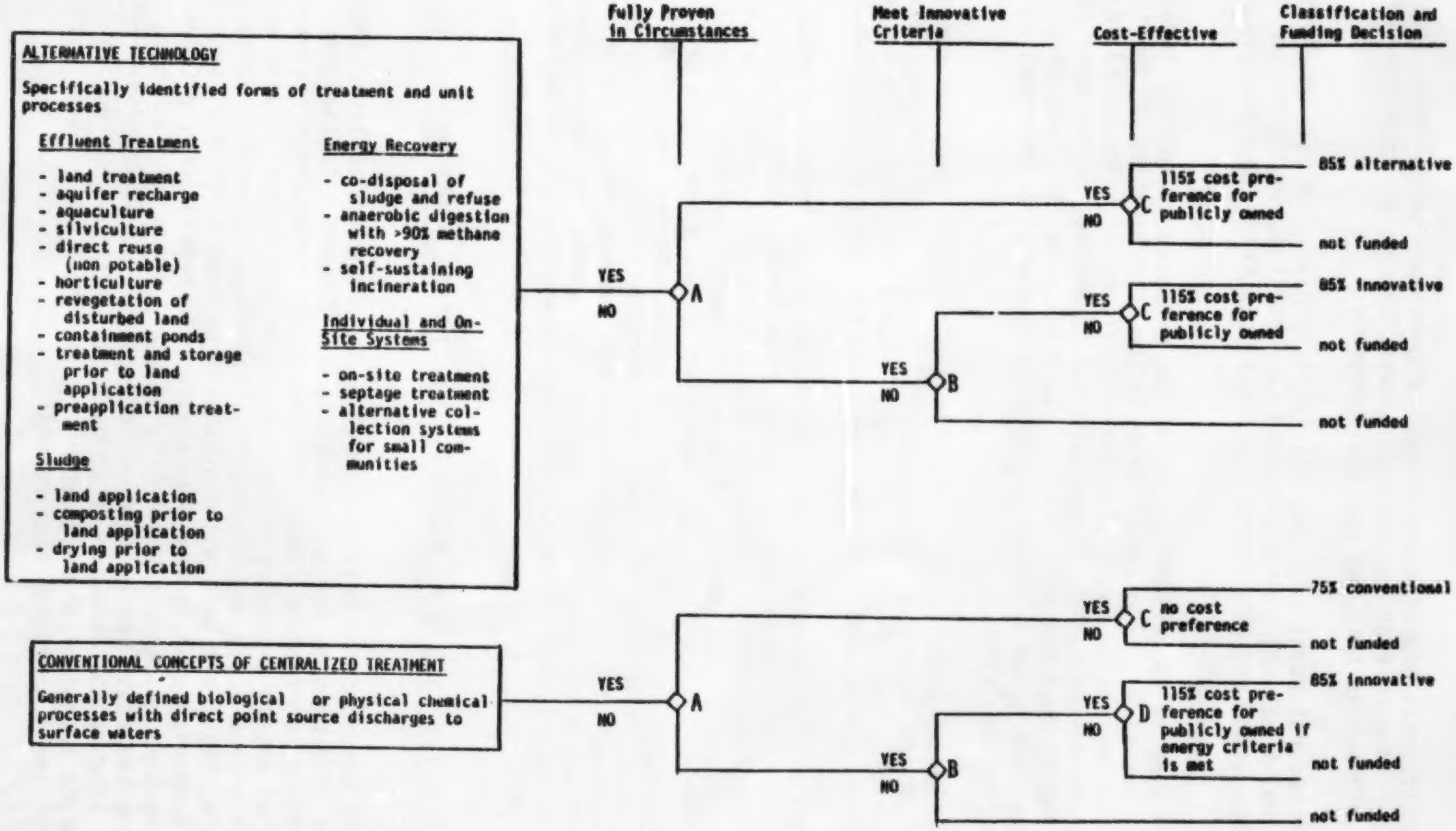

ALTERNATIVE TECHNOLOGY
Specifically identified forms of treatment and unit processes
Effluent Treatment
- land treatment
- aquifer recharge
- aquaculture
- silviculture
- direct reuse (non potable)
- horticulture
- revegetation of disturbed land
- containment ponds
- treatment and storage prior to land application
- preapplication treatment
Sludge
- land application
- composting prior to land application
- drying prior to land application
Energy Recovery
- co-disposal of sludge and refuse
- anaerobic digestion with >90% methane recovery
- self-sustaining incineration
Individual and On-Site Systems
- on-site treatment
- septage treatment
- alternative collection systems for small communities
CONVENTIONAL CONCEPTS OF CENTRALIZED TREATMENT
Generally defined biological or physical chemical processes with direct point source discharges to surface waters
Fully Proven in Circumstances
Meet Innovative Criteria
Cost-Effective
Classification and Funding Decision
YES
NO
A
B
C
D
115% cost preference for publicly owned
115% cost preference for publicly owned if energy criteria is met
no cost preference
85% alternative
85% innovative
75% conventional
not funded

FIGURE 3

INNOVATIVE AND ALTERNATIVE TECHNOLOGY DECISION METHODOLOGY

Response to the program from the design engineers has been mixed. They have approached the concept of innovative municipal designs with cautious optimism. Part of their reluctance is due to additional planning and engineering effort required for developing I/A Technology projects. The new evaluation procedures force all designers to learn a new set of design requirements and procedures which increase their non-billable costs while they have little assurance of recovering the added investment because of the interim nature of the presently authorized three-year program.

Other factors contributing to a slower-than-expected start of this program was the concern that the entire planning, design, and construction process may be slowed down to the point where increased costs due to inflation would outweigh the advantages of the I/A Technologies. Another concern on the part of designers is that during the last year of the three-year program, additional efforts to plan and design I/A projects would be wasted because the authorization to increase federal grants for construction would expire. Typically in the United States, facility planning (Step 1) and detailed designs (Step 2) costs average 7% of the total capital cost for municipal treatment plants. The normal time for planning and detailed design including state and federal approval range from 1 1/2 to 3 years depending on size and complexity of the facility and the level of public participation in resolving identified local environmental issues.

Notwithstanding the above concerns on the part of design engineers and the inherent time lags in implementing a new national program, results during the first 1 1/2 years have been very encouraging. Two hundred seventy-three facility plans have been received that include Innovative and Alternative Technology. Of these, 235 are for one or more of the 19 identified Alternative Technologies and 38 are for Innovative Technologies. The Innovative Technology facility plans are equally divided between proposed life cycle cost and energy saving designs.

The state governments' participation in the program has generally been supportive but all states have not yet established administrative procedures to fully implement the program because many were awaiting further clarification of procedure and policy direction from USEPA regional offices.

Because of the wide diversity of population density, geographical, and climatic features between the states, their ability to effectively benefit from the I/A Technologies differ markedly. The identified Alternative Technologies such as land application, aquaculture, silviculture, and other natural system technologies are more widely used in the midwestern and southwestern agricultural states with large, sparsely populated, potentially productive land areas. Improved centralized biological treatment plants, thermal sludge reduction procedures, and energy recovery and conservation systems are more widely used in the more populated eastern seaboard and Great Lakes areas. On-site technologies and the more cost effective alternative collection systems are used in all small communities (less than 3,500 persons) scattered throughout the U.S.

One of the immediately evident impacts of the I/A Program, even at this early stage, is a much closer link between the practicing engineers and the research community. Within USEPA, the Office of Research and Development and the Office of Water Program Operations are jointly participating in the program in an intensive and increasingly successful effort to develop and disseminate information on emerging Innovative and Alternative treatment approaches. Requests for information and assistance on new technological approaches have increased sharply during the past year. Another measure of increased interest in Innovative Technology is an increasing number of proposals for research demonstration and development grants from research communities and private industry.

A review of the I/A facility plans submitted thus far in the program indicates a wide range of technologies being considered by designers throughout the country. A partial listing of I/A projects representing different technologies, along with projected cost and energy savings, is shown in Table 4. These projects illustrate some of the newer technologies being proposed for municipal treatment in the U.S. Some have not yet been approved by the respective states or by the USEPA regional offices.

IV. SELECTED INNOVATIVE AND ALTERNATIVE TECHNOLOGIES

Summarized below are several examples, conceptual designs, treatment technologies, or techniques which demonstrate the broad scope and intent of the federal Innovative and Alternative Technology Program. They include both specific case histories and more general technology summaries. All are concerned with promoting national pollution abatement goals through increased emphasis on cost savings, energy conservation, improved application, or any combination of these. These examples were selected because they illustrate a wide range of technological and conceptual approaches proposed throughout the U.S. for solving local water quality problems. Some of the examples were found to meet the specific cost and energy criteria for Innovative Technology while others qualify as Alternative Technology.

In particular, there are nine summaries: Aquaculture and overland flow (part of land treatment) are discussed in general because they are promising Alternative Technologies which are expressly mentioned in the law as being desirable alternatives to conventional wastewater treatment. Unlike the proven slow rate and rapid infiltration land application systems, detailed design criteria for these systems are not as well developed. Thus, while potential benefits are great, these technologies still exhibit some risk. The combined sludge/solid waste disposal and energy recovery plant at Memphis, Tennessee is an example where an Alternative Technology also qualified as an Innovative Technology because of significant energy savings. The joint municipal and industrial waste facility proposed for Dodge City, Kansas is a good example of a conceptual design for a joint industrial municipal treatment facility which generates revenue, achieves maximum energy recovery, and recycles nutrients. Other elements of these projects include Alternative Technologies of aquaculture and land application. A combined aerobic/anaerobic

TABLE 4

COST AND ENERGY SAVINGS OF SELECTED INNOVATIVE OR ALTERNATIVE PROJECTS

Project Name or Location	Description of Technology	Innovative (I) or Alternative (A) Designation	Projected Savings Cost or Energy	Comments
Dodge City, Kansas	Joint anaerobic digestion of feed lot, packing house and municipal sludge lagoon, aquaculture treatment with sale of methane and fertilizer	I & A	Energy 25% Cost 10%	Comprehensive conceptual design for joint treatment with excellent reuse, recycle, and energy recovery features
Livingston, Montana	Windmill powered air activated sludge plant by using 5 40 Kw solar wind energy conversion system in small windmill form	I	Energy > 20%	May be co-funded by EPA and DOE power to be fed to network and treatment plant to draw power from same network
Kalamazoo, Michigan	Powdered activated carbon in single stage activated sludge with wet oxidation regeneration of carbon to treat highly industrialized municipal wastewater	I	Cost 16%	Large municipal plant 50 mgd
Montgomery, Alabama	Recovery of methane from anaerobic digesters, cleaning, compressing, and use to fuel sanitary department vehicles	A	N/A	First use of methane to fuel vehicles by municipal government under Construction Grants Program
Hardinsburg, Kentucky	Efficient use of large available hydraulic head at site to assist rotation of rotating biological contactors	I	Energy 36%	Plant design also incorporates other energy saving and recovery design features
Fountain Run, Kentucky	Hybrid system design for serving sparsely populated area. Includes on-site treatment by septic tank soil absorption, collection by small diameter gravity sewers and collection by pressure sewers and conventional sewers	A	N/A	One of the first examples of hybrid design and comprehensive planning using alternative collection and treatment technologies in the U.S. under the new Innovative and Alternative Technology Program

TABLE 4 (Continued)

Craig-New Castle, Virginia	Pressure sewer collection of septic tank effluent, solar pond heated anaerobic lagoon followed by facultative pond and fish aquaculture pond	A	N/A		Final design may differ from original proposal
Nassau County, New York	Short detention time fluidized bed biological treatment to meet secondary treatment standards	I	Cost Energy	15% 20%	This design does not meet the required cost or energy savings for innovative technology but has high potential and very wide applicability in the U.S.
Hillsborough, New Hampshire	Secondary treatment plant emphasizing rotating biological contactors. Plant is unique because of the total integrated energy saving design features that include solar anaerobic digester heating, energy efficient building design, heat recovery from the effluent and heated covered rotating biological contactors.	I	Energy	20%	Conceptually innovative design to maximize total energy savings thru the use of all feasible energy recovery concepts in an area of high energy costs.
Portland, Maine	Composting of municipal primary and secondary sludge and use for soil amendment on municipally owned land	A	N/A		None
York County, Pennsylvania	Aerated lagoon followed by land irrigation to meet secondary treatment standards	A	N/A		One of the more commonly used methods of land application in sparsely populated areas with suitable soils
Atmore, Alabama	Oxidation ditch (circular channel) method of activated sludge treatment with unique draft-tube aerator Aeration method uses U-tube principle for more efficient aeration using two prime movers that also have advantage of decoupling dissolved oxygen control and channel velocity	I	Energy	20%	A very reliable and energy efficient method of secondary treatment for small (0-10 MGD) plants. These systems also exhibit high potential for single stage nitrification-denitrification
Lorraine, Kansas	Three cell total containment lagoon	A	N/A		One of the more common forms of treatment for small communities in Midwest. Treatment is by evaporation and slow percolation

N/A--The cost and energy saving criteria are not applicable to alternative technology sludge digestion process proposed for Lackawanna, New York is a case history where formal evaluation of the cost and energy savings of the process qualified it as Innovative Technology. In another example, USEPA research to improve activated sludge aeration efficiency is summarized. A deep well biological treatment process for Ithaca, New York is representative of a new biological treatment approach which exhibits potential cost and energy savings. Finally, two examples of technologies showing promising Innovative characteristics which have been developed with partial USEPA research assistance are given. These are the vertical well chemical reactor system and fluidized bed biological treatment. The fluidized bed biological reactor was formally submitted and was recommended for approval as Innovative Technology. The vertical well chemical reactor system is at an early stage of development, but exhibits strong potential for cost and energy savings for sludge and higher strength wastes.

Vertical Well Chemical Reactor (8)(9)

Description

A vertical well chemical reactor (VWCR) system utilizes the fact that all organic substances can be oxidized in a liquid state if sufficient air (oxygen), temperature, and pressure requirements are met. Organic combustible elements are principally carbon and hydrogen and, in some cases, sulfur (when in sufficient quantities). Ideal combustion reactions for carbon and hydrogen including heat liberated are:

$$C + O_2 \rightarrow CO_2 \quad + 9.1 \text{ KwH/kg C } (14{,}100 \text{ BTU/lb C})$$

$$2H_2 + O_2 \rightarrow 2H_2O \quad + 39.4 \text{ KwH/kg } H_2 \ (61{,}100 \text{ BTU/lb } H_2)$$

In general, for most waste materials the range of heat released per pound of air required has been found to be from 0.35-0.41 KwH (1,200 to 1,400 BTU).

The objective of the VWCR system is to release this heat from the sewage or sludge and completely destroy all volatile elements while minimizing combustion imperfections and heat losses. Excess air quantities (above stoichiometric requirements) are required in varying amounts to take into consideration differences between ideal and actual conditions.

The principal feature of the VWCR system is its use of an extended vertical U-tube for achieving reaction pressures, temperatures, and retention time. The tubes can be suspended from the top of a conventinoal cased well and may extend to depths over 1,524 m (5,000 feet). The waste fluid and air are injected into a tube at the earth's surface. As the waste stream and air flow down the tube, they undergo natural pressurization due to the hydrostatic head above. Thus, water pumps and air compressors need only be nominally sized to overcome surface friction and gravity losses and do not need to develop the high pressures actually experienced at the bottom of the U-tube.

Two concentric tubes form the U-tube which is made of 300 Series stainless steel. The smaller downcomer tube is surrounded by hot treated waste from the larger concentric upcomer tube. Figure 4 is a schematic of the top view.

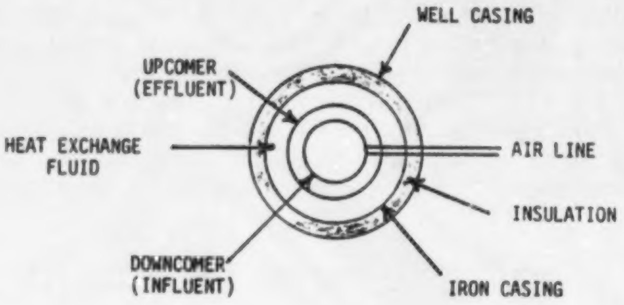


FIGURE 4

VERTICAL WELL CHEMICAL REACTOR SCHEMATIC, TOP VIEW

Heat exchange is very efficient and as influent waste approaches the bottom of the tube system, temperature and pressure become sufficiently high to support maximum oxidation. Pressures of over 1,500 psi and temperatures exceeding 600°F may be reached. Tube lengths and diameters are appropriately sized to allow adequate retention time (normally about 1/2 to one hour) in the oxidation/combustion zone. The well size estimated as necessary to treat sludge from a "typical" domestic wastewater treatment plant are as follows: a 55.9 cm (22 inch) diameter well for sludge from a .14 x 10^6 m^3/day (37 mgd) biological wastewater treatment plant; a 35.6 cm (14 inch) diameter well for sludge from a .075 x 10^6 m^3/day (20 mgd) plant; and 15.2 cm (6 inch) well for .019 x 10^6 m^3/day (5 mgd) and below.

Oxidized waste finally flows up the tube system and is gradually cooled as heat is transferred to the downflowing fluid. Pressure also decreases due to decreasing hydrostatic head as the fluid ascends. Thus, the pressurization and depressurization of the fluid waste and recouperative heat exchange are accomplished in a nearly reversible thermodynamic process. Excess heat is continually removed from the reaction zone by means of a heat exchange jacket and high pressure cooling water or other fluid. The high pressure, high temperature fluid is available for use at the ground surface as a high quality energy source.

The USEPA is currently sponsoring research on a pilot scale system designed to investigate the feasibility and cost of treating raw domestic sewage sludge by the VWCR oxidation process. It is felt the cost and energy savings gained by eliminating the expensive high pressure equipment used by conventional surface wet oxidation processes could be significant. The raw sludge being used has an input COD in the neighborhood of 3,000 mg/l. The research is being conducted at a pilot wastewater treatment plant in Denver, Colorado using a stainless steel reactor having a 5.1 cm (2 inch) diameter. Depth of the reactor into the ground is 457 m (1,500 feet). Approximately 22,740 l (6,000 gallons) of sludge will be processed daily. Oxidation efficiency as percent COD removal will be documented. Operational parameters and sludge dewatering characteristics will be measured as well as will selected analyses for nitrogen, phosphorus, organic, and heavy metal compounds. The study is scheduled to be completed in November, 1980.

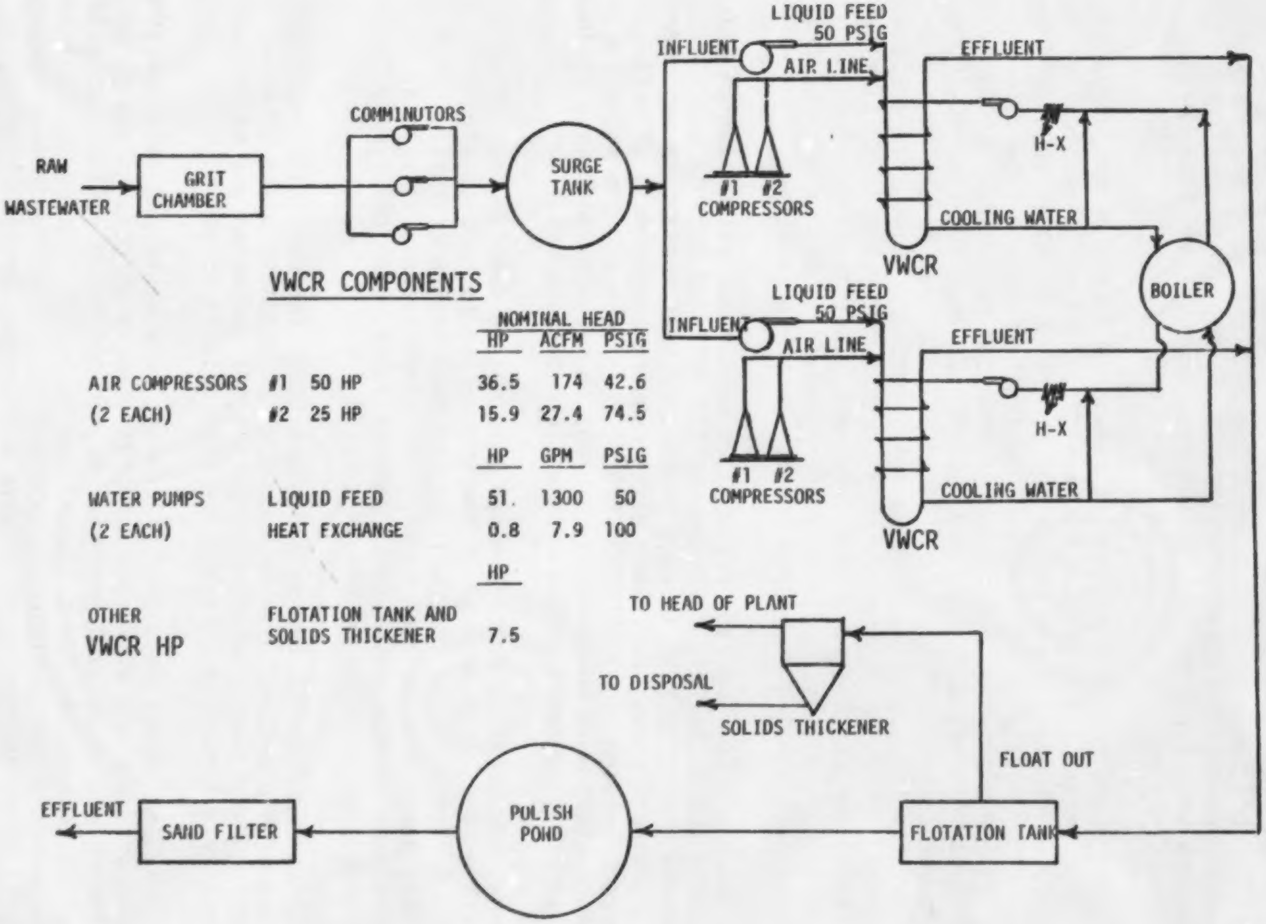
One example where the VWCR process is being considered as an Innovative unit process for treating a high strength municipal/industrial wastewater is at Montrose, Colorado where a new wastewater treatment plant is scheduled to begin construction in late 1980. Influent wastewater average flow is projected to be 13,230 m^3/day (3.75 mgd) and is composed of industrial (candy factory) and municipal wastes. Expected COD should range between 650-700 mg/l with a high sugar content which makes it desirable for wet oxidation. Preliminary calculations based on laboratory test results require 2 VWCR 1,524 m (5,000 feet) deep operating at temperatures up to 338°C (640°F). Preliminary layout and sizing for the Montrose VWCR treatment plant is diagrammed in Figure 5. Detailed analyses and final design considerations or options are, at this writing, still being developed for the Montrose facility.

Ideal conditions for the VWCR process, as with other wet oxidation processes, are when the exothermic reactions of its oxidized organic wastes provide adequate energy as heat to keep combustion self-sustaining and indeed to provide additional energy for other plant requirements. Most effective oxidation occurs between 260-302°C (500-575°F). In general, the waste (as sludge or wastewater) must be primarily organic in make-up and have a minimum COD of approximately 400 mg/l before exothermic reactions occur.

Efficient, recuperative heat exchange between the downflowing influent and rising effluent is also desired if the process is to be cost effective. In general, a ΔT of 1.5°C or less will be the VWCR design criteria. Once the surrounding earth equilibrates to reaction temperatures, it acts as an excellent insulator or heat envelope so that reliable, high efficiency transfer can reasonably be expected in the long term.

The VWCR U-tube uses 300 Series stainless steel for corrosion-resistant linings. More exotic metals could be required for specific waste streams. It has no moving parts and, therefore, incurs small operation and maintenance costs. A conservative design estimate for tube life is 15 years.

Actual cost and energy data are, of course, very waste and site specific. The heat value of the waste will vary and this greatly affects oxidation performance. The objective of VWCR operation is, as indicated earlier, to maintain its desired temperature for a long enough period of time so that all organic substances can be sufficiently oxidized to meet effluent requirements. (Pressure is fixed by reactor depth.) When exothermic conditions exist, as is expected most of the time, cooling water maintains the desired temperature range within the reaction zone and carries excess heat back to the surface for


RAW
WASTEWATER
GRIT CHAMBER
COMMINUTORS
SURGE TANK
INFLUENT
LIQUID FEED
50 PSIG
AIR LINE
#1 #2
COMPRESSORS
VWCR
EFFLUENT
H-X
COOLING WATER
BOILER
INFLUENT
LIQUID FEED
50 PSIG
AIR LINE
#1 #2
COMPRESSORS
VWCR
EFFLUENT
H-X
COOLING WATER
TO HEAD OF PLANT
TO DISPOSAL
SOLIDS THICKENER
FLOAT OUT
FLOTATION TANK
POLISH POND
SAND FILTER
EFFLUENT

VWCR COMPONENTS

		NOMINAL HEAD		
		HP	ACFM	PSIG
AIR COMPRESSORS	#1 50 HP	36.5	174	42.6
(2 EACH)	#2 25 HP	15.9	27.4	74.5
		HP	GPM	PSIG
WATER PUMPS	LIQUID FEED	51.	1300	50
(2 EACH)	HEAT FXCHANGE	0.8	7.9	100
		HP		
OTHER VWCR HP	FLOTATION TANK AND SOLIDS THICKENER	7.5		

FIGURE 5

PRELIMINARY MONTROSE VWCR TREATMENT SYSTEM LAYOUT other potential uses. When additional heat is needed such as for start-up or when the heating value of the waste drops below critical levels, it is supplied as steam by a pre-heat boiler.

The Montrose VWCR schematic demonstrates how these considerations are approached and illustrates the use of the heat exchanger and pre-heat boiler. Note also the use of the flow equalization tank preceeding the VWCR system which serves to dampen both hydraulic and organic slug loading. Similarly, the 12 acre-foot polishing pond also serves as an effluent damper and mixer, allowing final CO_2 to come out of solution and the pH to equilibrate with other organic acids formed (such as carbonic and acetic) which are among the final oxidation products. The Montrose VWCR system is being designed with 100% redundancy. One reason for its conservative design is because it is the first for a relatively large flow (13,230 m^3/day) and it is desired to insure continuous operation should some component require maintenance or modification. The dual configuration will also allow flow-splitting studies and long-term operation and maintenance data to be developed.

Laboratory and pilot scale data have given information about the degree of treatment which can reasonably be expected. COD removal should be between 80% to 90% or more. Effluent is, of course, disinfected and the solids, as an inert ash, are easily dewatered. It has been estimated that the volume of dewatered solids remaining after oxidation should be typically 0.11 m^3 (4 ft^3) per 3,780 m^3 (million gallons) of wastewater.

Table 5 shows the results of desk-top design calculations for treating domestic wastewater sludge from two sizes of wastewater treatment plants. Design assumptions are conservative and are indicated in the table. Cost and energy data are for the VWCR system proper (i.e., U-tube plus associated pumps and compressors, flotation tank, and solids thickener) and include well drilling costs.

TABLE 5

ESTIMATED VWCR SYSTEM COST AND ENERGY USE FOR TREATING SLUDGE AT TWO MUNICIPAL WASTEWATER TREATMENT PLANTS

	Nominal Plant Size	
	18.9×10^3 m^3/day (5MGD)	75.6×10^3 m^3/day (20MGD)
Capital costs, million dollars, $\$10^6$	2.5	5.0
Raw energy available, kw	556.4(1.9×10^6BTU/hr)	2196.2(7.5×10^6BTU/hr)
Energy consumed, kw	87.8(0.3×10^6BTU/hr)	263.5(0.9×10^6BTU/hr)
Operation, man years	3.5	3.5
Total pump power, kw	93.2 (125 hp)	261.0 (350 hp)
Reactor outer diameter, cm	15.2 (6 in)	35.6 (14 in)

*Assumptions: 0.24 kg COD/m^3 (2000 lbs COD/MG)
3.87 kwh/kg COD (6000 BTU/lb COD) heating value
75 percent recoverable heat, 1219.2 m (4000 ft) reactor depth
includes U-tube system, flotation tank, solids thickener

Innovative or Alternative Technology Characteristics

Although the thermodynamic principles involved in wet oxidation are well established, the VWCR system is a unique engineering application of these principles which has potential for significant energy recovery and cost savings compared to conventional unit processes which it might replace. Its desirable characteristics include three main areas. First, heat losses are minimized once surrounding earth is at equilibrium promoting efficient heat transfer and energy savings. This includes the potential for selling the energy (heat from the hot water) gained from the exothermic reactions. Secondly, its configuration promotes land conservation. It is estimated that the land required for a VWCR system (U-tube, pumps, compressors) to treat sludge from a 151,200 m^3/day (40 mgd) plant is approximately 2,023 m^3 (one-half acre). (Conversely, it should be noted that well drilling costs, if necessary, can be expensive.) Finally, the concept itself is extremely efficient because it takes advantage of natural hydrostatic head to create the pressures desired.

The use of natural forces to attain a desired result means potentially more reliable operation as well as cost savings compared to the expensive high pressure equipment normally used. Cost savings are also potentially available as a result of the natural advantages of the wet oxidation process. These include its ability to oxidize strong waste liquors and sludges without pretreatment, and the inert, easily dewatered sludge it produces.

While the VWCR system has significant potential, formal data acquisition has only recently begun at pilot scale (July 1979) and the process has yet to be operated full scale. Thus, the technology is not fully developed and there is some risk involved. High temperatures and pressures promote accelerated corrosion or deposition when conditions and water constituents are at the right levels. There is some possibility of plugging the U-tube reactor. Start-up and shut down procedures, operation and maintenance protocols, mechanical and thermodynamic problems in large-scale operations still have to be addressed. These represent an engineering challenge to a technology which can potentially meet national environmental goals of increased energy recovery and improved cost effectiveness.

Improved Aeration Efficiency and Reliability (10)(11)

Description

The Innovative and Alternative Technology Program administered by the USEPA tangibly rewards promising technologies intended for full-scale treatment which have a reasonable chance of effecting significant cost or energy savings, resource conservation or reuse, or public benefits. It would be foolish, however, to expect these technologies to emerge from a vacuum, and no such pretense is made. USEPA wastewater research efforts are oriented towards enhancing development and management of wastewater treatment technologies or techniques which have a high potential for significant beneficial impact on wastewater treatment objectives. Attention is not limited to treatment facilities alone. It includes scrutiny of the approaches and bases for developing, selecting, and optimizing the unit processes or unit operations used.

One example is USEPA recognition of the need for an oxygen transfer standard method of measurement in municipal wastewater. There are two common complaints about oxygen transfer equipment. These are: (1) difficulty in correlating manufacturers' claims with field test data; and (2) difficulty in projecting oxygen transfer efficiencies from clean to wastewater systems. As a result, discrepancies between anticipated and actual field conditions often necessitate substantial field modifications to the aeration equipment furnished.

Table 6 illustrates clean water oxygen transfer efficiencies for several generic aeration devices under standard conditions. Noting the uncertainties created when trying to translate clean water oxygen transfer efficiencies to field conditions, R.C. Brenner wrote, "It is quite evident that if the claimed oxygen transfer capabilities of fine bubble aeration systems in particular can be substantiated (for municipal wastewaters) via repeated testing using valid standardized test and evaluation methodology and if the maintenance requirements of these devices are not exorbitant, substantial savings in power are possible in contrast to the traditional, widely used coarse bubble options" (11). As a result of this recognition, USEPA has undertaken research in three related areas.

From August 1977 to February 1979, the Los Angeles County Sanitation Districts (LACSD) supported in part by USEPA, conducted a series of clean water oxygen transfer tests designed to obtain data under closely controlled conditions comparing various generic types of submerged air aeration equipment. Six types of devices were selected which represented typical methods of dispersing various sizes of bubbles:

Fine Bubbles	Medium Bubbles	Coarse Bubbles
Jet Aerators	Static Aerators	Fixed Orifice
Dome Diffusers		Variable Orifice
Tube Diffusers		

All devices were compared using the same test tank and identical test procedures throughout. The test tank was 6.1 m x 6.1 m x 7.6 m (20 ft x 20 ft x 25 ft) maximum variable depth. Clean water dissolved oxygen uptake was carried to equilibrium using water chemically deoxygenated with sodium sulfite and a cobalt chloride catalyst. Three runs at four different depths (3.0, 4.6, 6.1 and 7.6 m) were made. Each run had a different input power level (varying from .01-.04 Kw/m^3) delivered to the water. Diffuser configurations were selected by aerator manufacturers who were allowed different configurations for different depths but had to maintain a constant configuration over the series of tests at any given depth. The configuration or geometric pattern selected was one intended to be economically feasible at full scale and over the range of input powers evaluated.

TABLE 6

COMPARATIVE CLEAN WATER OXYGEN TRANSFER INFORMATION FOR AIR AERATION SYSTEMS UNDER STANDARD CONDITIONS (a)

Type of Aeration Device	Range of Clean Water O_2 Transfer (%)	Range of Clean Water Efficiencies kg O_2/kwh (lb O_2/hp-hr)	Energy Requirement kwh/kg O_2 (kwh/lb O_2)
Mechanical Aerator			
Low speed surface	-	1.5-2.2 (2.5-3.5)	0.46-0.66 (0.21-0.30)
High speed surface	-	1.2-1.8 (2.0-3.0)	0.55-0.82 (0.25-0.37)
Turbine sparger (b)	14-18	1.2-1.8 (2.0-3.0)	0.55-0.82 (0.25-0.37)
Fine Bubble Aerators (c)			
Fine Bubble Diffuser			
(a) Total floor coverage	20-32	3.0-4.6 (5.0-7.5)	0.22-0.33 (0.10-0.15)
(b) Side wall mounted	15-20	1.8-3.3 (3.0-5.5)	0.31-0.55 (0.14-0.25)
Jet Aerator (b)	15-26	1.6-2.3 (2.7-3.8)	0.44-0.62 (0.20-0.28)
Coarse Bubble Diffuser (c)			
Static aerator	10-16	1.4-1.9 (2.3-3.2)	0.51-0.71 (0.23-0.32)
Coarse bubble dual aeration	10-13	1.4-1.6 (2.3-2.7)	0.62-0.71 (0.28-0.32)
Coarse bubble single side aeration	8-10	1.2-1.5 (2.0-2.5)	0.60-0.71 (0.30-0.32)

(a) Compiled using a combination of manufacturers' company bulletins, technical reports, and historically accepted data ranges.
(b) Includes energy requirements for two prime movers.
(c) Based on clean water test at 15 ft water depth; submergence varies depending on device.

Recognizing that the ultimate test of any aerator performance is over the long term under actual field operating conditions, a 2 1/2 year effort is scheduled at LACSD which will evaluate the three most promising aeration devices as determined by the above clean water tests. Not surprisingly, the three fine bubble aeration devices have been verified as most promising, relative to potential power savings. Evaluation will begin in May of 1980 after the three aeration systems have been installed in three parallel activated sludge aeration tanks at the LACSD Whittier Narrows Wastewater Treatment Plant. This plant receives an average flow of 56,700 m^3/day (15 mgd) at an average influent BOD equal to 310 mg/l, average COD equal to 640 mg/l, and average suspended solids equal to 390 mg/l.

Finally, to partially offset the informational vacuum that now exists in the United States and most likely would continue to exist until results of the LACSD field study are finalized, a year-long study of data and experiences from energy-efficient air aeration activated sludge treatment plants in other countries has been commissioned by USEPA. Europe, and especially Britain, has had considerable experience with fine bubble diffusers in its activated sludge plants. Assessment and review of operating experiences and data from 13 European treatment plants is intended to fill the data gap while more extensive data are being generated in the United States. The study, which is nearing completion, will analyze and summarize timely and useful operating data from full-scale plants having a 4-8 year data base using fine bubble diffusers. The final report is expected to be available by October 1980.

As indicated previously, the clean water testing by the LACSD was done using the non-steady state reaeration method. Considerable effort was made to keep constant all external variables so that differences in aerator performance could be attributed with confidence to inherent limitations and not outside influences. Generally speaking, the fine bubble aeration systems showed a significant improvement over coarse bubble spargers. Aerator transfer efficiencies and energy consumption under the varying conditions described earlier were measured and documented to include information such as that illustrated in Table 6. The final report on this effort has not been completed at this writing, but is expected to be published shortly and will contain detailed data about the various aerator performances.

Completion of field tests on porous dome diffusers, porous tube diffusers, and jet aeration systems at LACSD's Whittier Narrows Wastewater Treatment Plant is ex pected by early 1982. Data gathered over a two-year period is intended to supply needed information for at least three areas which lack an adequate data base for good cost and energy estimations. First, long-term operational and maintenance problems of fine bubble aeration devices such as air filtering requirements, external diffuser clogging, and diffuser breakage will be documented and analyzed. Secondly, oxygen transfer efficiencies in respiring full-scale activated sludge systems subject to real-world influences will be closely and comprehensively measured over time. Finally, the potential for developing an efficient, high-rate activated sludge process (i.e., short detention times, high organic loadings) with fine bubble aeration systems will be explored. The high volumetric organic loadings utilized by high-solids, high-rate systems can be sustained only by efficient oxygen transfer such as provided by use of pure oxygen aeration or hopefully by fine air bubble production. Decreased bioreactor volume and energy savings would be the by-products of a successful study.

The LACSD effort will also include extensive measurements to determine alpha and beta factors under field conditions. These factors, particularly alpha, significantly affect aeration equipment field design. Alpha is the ratio of the overall oxygen transfer rate coefficient in wastewater to that in clean "tap" water. Alpha is known to vary considerably for different wastewaters, tank geometrics, and bubble sizes. Beta is the ratio of the wastewater system dissolved oxygen saturation concentration to that in clean water. These tests will be conducted in the same mixed liquor tanks as the performance evaluations, affording the unusual opportunity to measure alpha and beta factors on a plant rather than laboratory scale.

The survey and evaluation of existing energy efficient air aeration devices in other countries is intended to provide timely maintenance information and operating data which will beneficially impact use of fine bubble aeration equipment in existing and future activated sludge plants. The survey will define and summarize international professional trade literature with respect to aerator availabilities and process and maintenance performance. It will document and assess records from actual operating plants with a good data base and sufficient history ($\geq$4 years) to provide a reliable indication of actual aerator processes and maintenance performance. Finally, it will address wherever possible points of controversy concerning the efficiency of various aerator types where good quality data can be developed on specific points. The resulting report should provide USEPA and the industry with an early and definitive report about fine bubble aerator performance in operating, full-scale activated sludge systems.

Innovative or Alternative Characteristics

The underlying concept of the Innovative and Alternative Guidelines is to provide incentive for widespread adoption of municipal wastewater treatment technologies or techniques which represent advancement in the current state of the art with respect to meeting the national environmental goals set out by Congress. Energy efficiency and cost effectiveness are two of these goals met by improved oxygen transfer. In particular, the long-term benefits obtainable from more efficient aeration devices have been summarized by Brenner and represent a case-in-point which demonstrates desirable Innovative characteristics: confidence in the efficiency and field performance of fine bubble wastewater aeration devices should result in more efficient high-rate activated sludge systems with a concomitant savings in energy over design techniques to avoid unintended undersizing of oxygen transfer equipment.

Deep Well Biological Reactor (12)(13)

Description

A deep well biological reactor (DWBR) is basically a vertical activated sludge reactor which has the potential to reduce land area required for treatment, life cycle costs, and energy requirements. Treatment of raw wastewater occurs at elevated pressures with turbulent mixing promoting efficient oxygen dissolution and intimate biota/substrate contact. Figure 6 illustrates schematically the DWBR configuration.

Influent raw wastewater and sludge recycle are fed to the head tank located directly above the DWBR, where it combines with return sludge. The DWBR is divided into an upflow section (called the riser) and a downflow section (called the downcomer). The primary purpose of the head tank is to allow spent air to disengage from the mixed liquor exiting the riser, prior to re-entry into the downcomer.

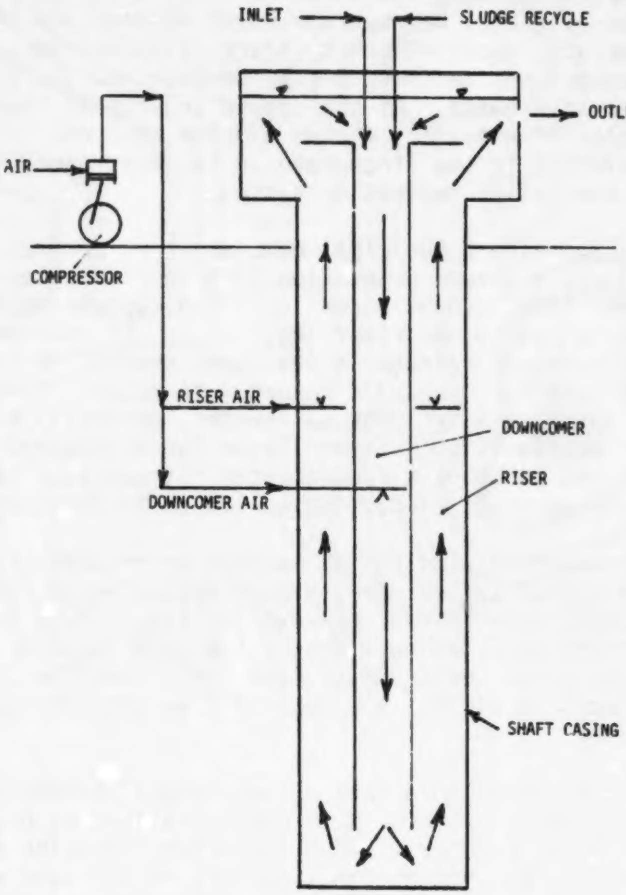

INLET
SLUDGE RECYCLE
OUTLET
AIR
COMPRESSOR
RISER AIR
DOWNCOMER
RISER
DOWNCOMER AIR
SHAFT CASING

FIGURE 6

DWBR HYDRAULICS

The initial circulation of the mixed liquor in the shaft is induced by a simple air lift pump principle using compressed air injected into the riser side of the shaft. Once shaft circulation is established, the air supply is gradually transferred to the downflowing side.

Since the veolicity of the rising bubbles injected into the downcomer is 4-7 times less than the liquid velocity in the downcomer, these bubbles are carried downward to the bottom of the shaft. In many designs, a large portion of the bubbles will be completely dissolved before they reach the bottom of the shaft.

The driving force causing circulation is created by the net difference in total bubble volume (voidage) between the riser section and in the downcomer section. Above the air injection point, there is no voidage. Therefore, the liquid in the downcomer has a lower average voidage and its greater weight forces the flow down the shaft. As the liquid travels up the riser, pressure decreases and bubbles of nitrogen, carbon dioxide, and residual oxygen are formed and then released to the atmosphere in the head tank. Treated effluent then overflows to the solids separation system.

The USEPA is supporting a municipal demonstration project at Ithaca, New York. It utilizes a second generation DWBR configuration and began operation in October 1979. In this configuration, approximately 2/3 of the compressed air is fed to the riser leg, 1/3 to the downcomer leg. The technique greatly increases voidage in the upper section of the riser making it virtually impossible for hydraulic reversal to occur. Further, virgin air is introduced along with influent wastewater, optimally matching oxygen demand with oxygen availability. Mixed liquor for subsequent clarification is withdrawn below the point of air/wastewater introduction to prevent untreated wastewater from short circuiting to the shaft discharge line.

Ithaca will be investigating solids separation of DWBR effluent using two modes. In the sedimentation mode, vacuum degassing to remove additional effervescence preceeds conventional gravity settling. MLSS in the reactor are usually restricted to 5,000 mg/l due to the settling characteristics of the mixed liquor solids in the sedimentation tank. Recycle sludge thickens to roughly 10,000 mg/l resulting in a recycle flow rate of approximately 100% of influent flow.

The flotation mode takes advantage of the natural tendency of the mixed liquor leaving the riser to float. MLSS concentrations as high as 10,000 mg/l can be maintained in the reactor without impairing flotation solids separation efficiency. A smaller DWBR volume can therefore be utilized while retaining F/M levels in the design range of 0.75-1.0. Solids in the float can vary from 6-8% eliminating the need for further thickening and allowing a small float recycle flow rate of about 20%. Flotation aids may be necessary, however, offsetting savings in bioreactor size and sludge disposal costs. Schematics of the DWBR in both sedimentation and flotation modes are illustrated in Figures 7 and 8.

The Ithaca municipal demonstration plant DWBR is 136 m (446 feet) deep and has a main steel casing of 43.82 cm (17.25 inches) ID grouted to the geological formation with cement. The primary downcomer is 29.85 cm (11.75 inches) OD. Compressed air is added to both the downcomer (approximately 0.566 standard m^3/min) and the riser (approximately .934 standard m^3/min) using a 14.9 kw (20 hp) compressor. The DWBR acts as a plug flow bioreactor. Biological growth and substrate utilization proceeds in accordance with the Michaelis-Menten relationship. Operation is based on an average BOD influent of 150 mg/l. The combination of high intensity mixing in the shaft (Reynolds Numbers $\geq$ 100,000) and elevated pressures (to 8 atm) would produce oxygen utilization efficiencies in excess of 90% with an influent BOD of 500 mg/l or more. However, with the relatively weak Ithaca influent, circulation air requirements govern and oxygen utilization is only expected

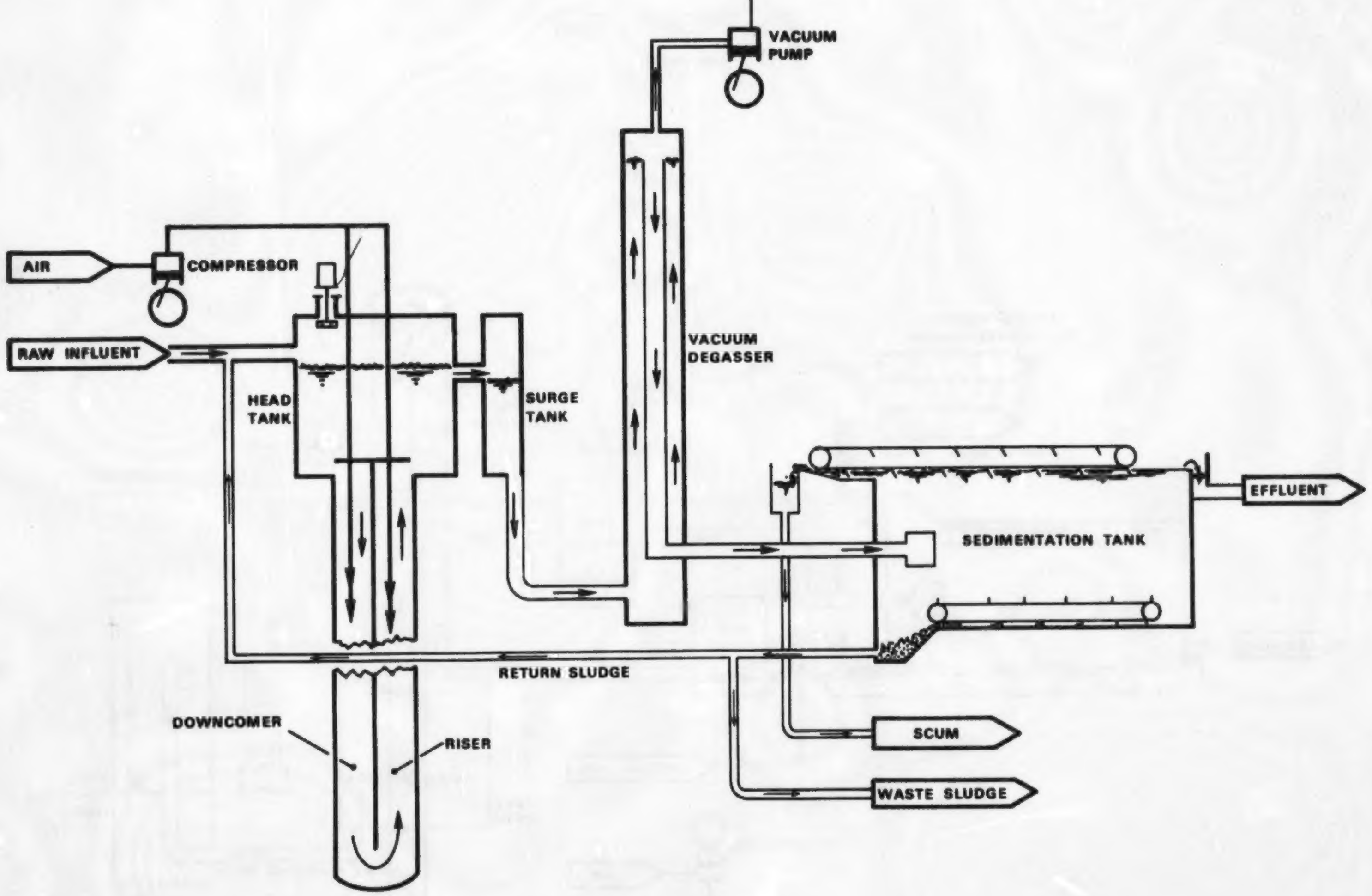

VACUUM PUMP
AIR
COMPRESSOR
RAW INFLUENT
HEAD TANK
SURGE TANK
VACUUM DEGASSER
SEDIMENTATION TANK
EFFLUENT
RETURN SLUDGE
DOWNCOMER
RISER
SCUM
WASTE SLUDGE

FIGURE 7

DWBR SEDIMENTATION MODE

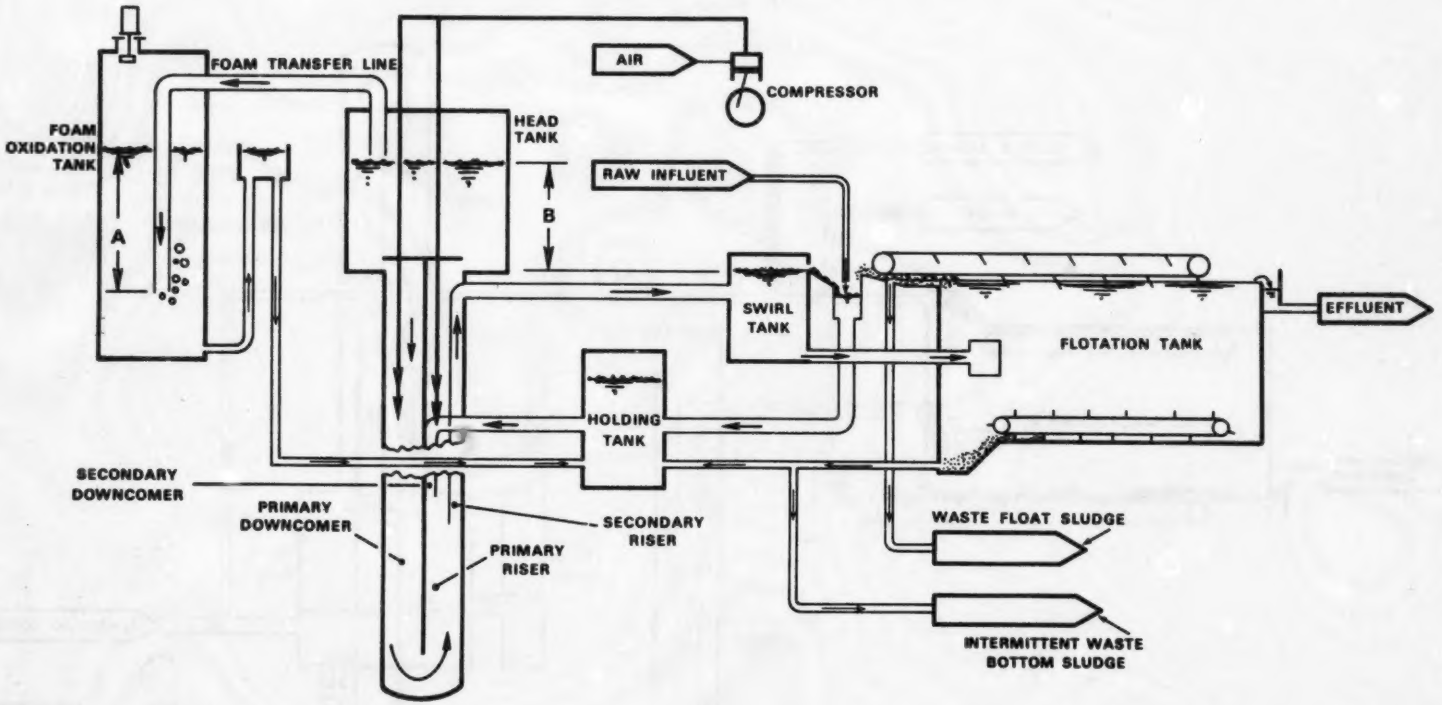

FOAM TRANSFER LINE
FOAM OXIDATION TANK
A
HEAD TANK
B
AIR
COMPRESSOR
RAW INFLUENT
SWIRL TANK
FLOTATION TANK
EFFLUENT
HOLDING TANK
SECONDARY DOWNCOMER
PRIMARY DOWNCOMER
PRIMARY RISER
SECONDARY RISER
WASTE FLOAT SLUDGE
INTERMITTENT WASTE BOTTOM SLUDGE

FIGURE 8

DWBR FLOTATION MODE

to reach 40-45%. Biological sludge production for the Ithaca plant is expected to be equal to or less than 0.6 kg biological solids/kg BOD removed at F/M loadings of 0.75-1.0. Operational parameters to produce secondary treated effluent (BOD/SS as 30/30 mg/l) are summarized in Table 7. Effective operation of the plant will depend on controlling MLVSS within a predetermined F/M range over time.

TABLE 7

DWBR OPERATING PARAMETERS

Parameter		Operational Mode	
		Flotation	Sedimentation
DWBR			
Average Influent Flow,	m^3/day	757	379
	MGD	0.2	0.1
MLSS,	mg/l	10,000	5,000
SRT,	days	5.7	5.7
F/M[1],	day^{-1}	0.74	0.74
Volumetric Loading[2],	kg BOD /day/m^3	5.54	2.77
	lb BOD /day/1000 ft^3	346	173
Shaft Detention Time[3],	minutes, nominal	39	78
	minutes, actual	24	39
Recycle Flow Rate,	bottom solids recycle, percent	40	
	float solids recycle, percent	20	100
Solids Separation Unit			
Overflow Rate,	m^3/day/m^2	20.1	10.1
	GPD/ft^2	494	247
Mass Loading[4],	kg TSS/day/m^2	321	103
	lb TSS/day/ft^2	66	21
Return Sludge TSS,	percent	(bottom solids) 2-3	
		(float solids) 6-8	1-2
Waste Activated Sludge[5],	kg TSS/kg BOD removed	0.6	0.6
	kg TSS/day	54	27
	lb TSS/day	120	60

1. F/M ratio expressed as influent BOD in kg/day and MLVSS under aeration in kg assuming an influent BOD concentration of 150 mg/l and mixed liquor VSS of 75 percent.
2. Volumetric loading estimated assuming influent BOD of 150 mg/l and VSS of 75 percent, a nominal shaft diameter of 438 mm and a shaft depth of 136 m.
3. Actual detention time based upon sludge return recycle rates of 100 percent in the sedimentation mode and 20 percent and 50 percent, respectively, for float solids and bottom solids in the flotation mode.
4. Solids separation unit mass loadings are based upon sludge return rates of 100 percent and 60 percent MLSS concentrations of 5,000 and 10,000 mg/l, respectively, in the sedimentation and flotation modes.
5. Activated sludge wasted per unit of BOD removed based on an influent BOD of 150 mg/l and an effluent BOD of 30 mg/l.

Innovative or Alternative Technology Characteristics

The Ithaca municipal demonstration plant project is intended to investigate the feasibility of DWBR technology and gather potential scale-up data for building a full-scale plant to treat its wastes. Average flow at Ithaca is approximately 37,800 m^3/day (10 mgd) with peak flow expected at 93,366 m^3/day (24.7 mgd). In a full-scale DWBR facility at Ithaca, high grade waste heat would be available from its two proposed 112 kw (150 hp) compressors (224 kw total) and could be used to heat an enclosed process facility. Heat recovered from the compressor oil lubricant at 65.5-71.0°C (150°-160°F) cooled to 48.9°C (120°F) yields 164 kw (560,000 BTU/hr).

Additional savings will occur if, as may be the case for Ithaca, chemical flotation aides can be eliminated. Highly concentrated waste sludge, space economy, and lack of moving parts in the subsurface shaft all contribute to potentially lower costs. Pilot plant efforts at Ithaca are directed towards establishing what actual net cost and energy savings can be expected as well as providing definitive scale-up data. Items which will be addressed include:

1. The desirability of operating in the flotation or gravity mode.
2. The effect of flow and organic loading variations.
3. The exact quantity of chemicals, if any, required to aid flotation.
4. The effect of chemical addition to primary effluent for phosphorus removal.
5. The exact quantities of sludge produced by the system.
6. Full scale power requirements.
7. Full scale equipment configuration: the cost and feasibility to operate a single shaft with two flotation tanks compared to two shafts with one flotation tank.

Joint Municipal and Industrial Waste Treatment (14)(15)

Description

In order to meet National Pollution Discharge Elimination Permit conditions, Dodge City, Kansas, a midwestern city discharging wastewater to the Arkansas River, considered several treatment alternatives to upgrade and expand its existing treatment facilities. Significant in their evaluations was the fact that its wastewater contained a large quantity of high strength meat packing wastes (3,400 of 15,700 m^3/day). The alternatives evaluated included trickling filter followed by rotating biological contactor (RBC) treatment, and different combinations of land application and joint wastewater treatment. This last alternative was possible because a privately-owned biogas conversion plant utilizing and processing manure from cattle feed lots was interested in treating some or all of the Dodge City's wastewater. The private plant produced methane and fertilizer for resale from its biogas conversion facilities. The city could share in methane gas revenue. Furthermore, the private plant routed its solids dewatering centrate to aquaculture basins whose effluent could be used for crop irrigation. The alternatives considered are summarized in Table 8.

TABLE 8

TOTAL NET PRESENT WORTH COMPARISON OF ALTERNATIVES AS SHOWN IN THE DODGE CITY FACILITY PLAN

	Treatment Alternatives Considered	Net Total Present Worth*	Comments
C	Trickling filter plus RBC treatment and discharge	$6,329,800	Most cost effective non-innovative alternative
E	Complete treatment by privately-owned plant; wastewater to crop irrigation	6,927,700	Not including irrigation revenue of $62,200/year
F	Wastewater treatment and discharge with sludge and meat packing flow to privately-owned plant	5,689,00	Proposed as innovative in application
G	Land application of entire flow on city-owned site	8,953,100 (7,279,270)	Are alternative technology and qualify for 15% cost effectiveness preference
H	Seasonal land application	7,518,100 (7,279,270)	Are alternative technology and qualify for 15% cost effectiveness preference

*1978 dollars
( ) 115 percent of $6,329,800

The facility plan describing the alternatives in detail was submitted to USEPA in accordance with the new national I/A provisions. A preliminary cost analysis made to determine if joint treatment could qualify as Innovative Technology identified alternative F (Table 8) as the lowest cost Innovative Technology with baseline non-innovative technology being alternative C. Note that Alternative Technologies E and H (land application) did not meet 115% cost criteria over alternative C and thus were eliminated from further consideration. Alternative F also did not meet life cycle cost criteria to qualify it as Innovative Technology. However, closer inspection (detailed below) suggested it could qualify under net primary energy criteria.

Alternative F falls under the classification of conventional concepts of treatment and must meet either a 15% life cycle cost reduction or a 20% energy reduction over its nearest non-innovative competitor in order to be classified Innovative Technology. As noted previously, a preliminary analysis did not indicate 15% cost savings (although cost savings were close enough so that final cost re-evaluations prior to the final design may save 15% or more). A net primary energy analysis did show energy savings above 20%. A summary of the energy analysis is summarized in Table 9. Alternative F was subsequently recommended for additional I/A funding, however it was not adopted because of local environmental factors. It is included here to demonstrate the excellence of the concept.

TABLE 9

NET PRIMARY ENERGY ANALYSIS FOR ALTERNATIVES C AND F FOR THE DODGE CITY FACILITY PLAN

Alternative C @ Q=15687 m^3/day (4.15 MGD)	Net Primary Energy kwh/year
Raw wastewater pumping	252,849
Head works	77,010
Primary clarifier and sludge pump	14,632
Flow equalization basin	11,295
Recycle and equalization pumping station	508,266
RBC units	550,365
Effluent reaeration basin	20,279
Solids disposal	45,340
Subtotal	1,480,306
Net digester energy produced	(-520,000)
Net primary energy	960,306
Alternative F @ Q=12285 m^3/day (3.25 MGD)	
Raw wastewater pumping	197,195
Plant head works	77,010
Building heat	14,375
Re-equip existing primary clarifier	11,551
Flow equalization basin	8,727
Recycle equalization pump station	397,885
RBC units	341,411
Effluent reaeration	15,915
Hy Plains and sludge pump station	156,587
Standby effluent pumping	7,187
Standby power	5,647
Electrical control center	7,187
	1,240,677
Net equivalent exported energy	(-520,000)
Net primary energy	720,677

$$\text{Percent Net Primary Energy Savings} = \frac{(960,306 - 720,677) \times 100}{960,306} = 24.9\%$$

Innovative or Alternative Technology Characteristics

In addition to the cost and energy savings estimated above, the following considerations contributed to the overall Innovative Technology assessment of Dodge City's wastewater facility plan:

1. The unique, mutually beneficial arrangement between a municipality and private industry for the purpose of wastewater treatment.
2. Maximum recycle and reuse of the industrial wastewater and domestic wastewater sludge for fertilizer, energy production, and irrigation.
3. The use of a revenue-generating complex.

There were also some risks recognized as associated with the Innovative Technology approach proposed:

1. Operation of the plant is dependent upon the privately-owned conversion plants continued financial and technical stability.
2. Use of the solids dewatering centrate as feed to the aquaculture system is not proven technology. Some pretreatment, such as dilution, will have to be explored.

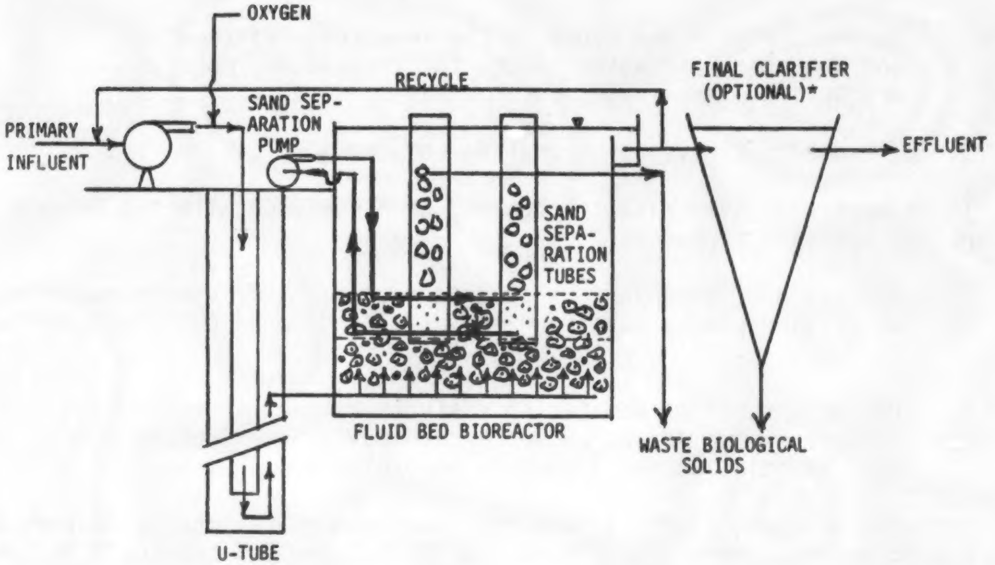
In spite of these risks, it was felt that the costs, energy conservation, and public benefits were significant enough that the project could be considered Innovative in its approach and qualify for additional federal funding.

Fluidized Bed Biological Treatment (16)(17)

Description

Nassau County, New York is presently designing additional secondary wastewater treatment facilities to upgrade and expand their existing Bay Park Wastewater Treatment Plant from .23 to .265 x 10^6 m^3/day (60 to 70 mgd). Expansion facilities will utilize fluidized bed biological treatment (FBBT). This design is another example of an improved biological treatment process submitted to USEPA for increased grant funding under the new National I/A Program. It was recommended for funding.

The FBBT process consists of a columnar bioreactor partially filled with fine grained media such as carbon or sand having an effective size approximately 0.6 mm and a uniformity coefficient of 1.4. Primary effluent is passed up through the bottom with enough velocity ($>$ .01 m/sec) to expand or "fluidize" the media. The media acts as a support surface upon which a firmly attached biomass eventually grows and thrives. Fluidizing the media effectively increases pore volume because the particles are not in contact. Flow through the fluidized bed is laminar even though all particles are suspended and kept in motion. As a result, intimate contact of the entire surface area is assured giving greater opportunity for the biomass to effectively consume wastewater contaminants. Oxygen is provided by a 61 m (200 feet) deep oxygen fed U-tube reactor. Reactor effluent at the Bay Park plant passes through a conventional secondary clarifier before discharge. A diagram of the fluidized bed reactor using a U-tube aerator is shown in Figure 9.


OXYGEN
RECYCLE
FINAL CLARIFIER (OPTIONAL)*
PRIMARY INFLUENT
SAND SEPARATION PUMP
EFFLUENT
SAND SEPARATION TUBES
FLUID BED BIOREACTOR
WASTE BIOLOGICAL SOLIDS
U-TUBE

* ALTERNATIVE SOLIDS SEPARATION DEVICES ARE UNDER CONSIDERATION. EXISTING FINAL CLARIFIERS ARE PROPOSED TO BE USED AT BAY PARK WASTEWATER TREATMENT PLANT.

FIGURE 9

FBBT REACTOR SCHEMATIC

Advantages of the trickling filter and the activated sludge processes are utilized in the FBBT process. As in trickling filters, the reactor requires no recirculation to maintain high concentrations of biomass. Furthermore, pilot reactors have resulted in MLVSS > 14,000 mg/l compared to typical conventional activated sludge MLVSS concentrations of 1,500 mg/l. As a result, treatment time and bioreactor volume are reduced. Pilot plant evaluations using wastewater from the Bay Park Wastewater Treatment Plant have indicated that with a recycle ratio of 1.5, satisfactory secondary treatment of its primary sewage is possible in as little as 15 minutes pilot reactor empty bed detention time compared to the several hours required for conventional activated sludge treatment. Recycling is necessary because dissolved oxygen content is limiting. It has not been possible to add sufficient oxygen to the wastewater in a single pass to support the biological oxidation that occurs in the high MLVSS reactor.

Expanded bed height and particle size control (biological slime thickness) is maintained by automatic ultrasonic controllers that operate a proprietary sand-separation pump which keeps sludge height between set limits. Separated sand is returned to the bed and sludge floc continues to final clarification. Another system option is to withdraw waste solids from internal sand separation tubes thus eliminating the necessity of a secondary clarifier. The sludge removed has a 1-3% solids concentration and is approximately 0.1-0.3% by volume of the forward flow. The relatively high sludge solids concentration and low sludge flow reduce sludge handling costs considerably.

Considerable effort has been made investigating methods and techniques to dissolve large amounts of oxygen in Bay Park wastewater since FTTB feasibility studies began in the latter part of 1974. Early studies determined that pre-dissolved oxygen in the wastewater was desired over direct injection because of the undesirable turbulence and effervescence created in the fluidized bed reactor by the latter operation. Different methods of oxygen dissolution were explored. These included an aeration cone device which was designed to provide long contact time between the oxygen gas and water, and a pressure swing oxygenating device which incorporated wastewater transfer pumps in its pressure loop eliminating the need for them to work against a pressure head. The most successful method to date has been an oxygen-fed U-tube used to add dissolved oxygen to the incoming wastewater before entering the reactor. The technique forces the liquid and dissolved gas down the U-tube. Natural hydrostatic head increases the pressure as the mixture travels downward. An increase in dissolved gas deficit and thus an increase in gas transfer results. The U-tube configuration used, unlike that the name suggests, has two concentric tubes where the gas/liquid mixture is pumped down the middle smaller one and the dissolved gas mixture flows up the larger. Design criteria and preliminary FBBT and U-tube specifications are summarized in Table 10.

TABLE 10

PRELIMINARY DESIGN CRITERIA AND REACTOR SIZING FOR THE 10 MGD EXPANSION OF THE BAY PARK WASTEWATER TREATMENT PLANT

Average Design Conditions

Influent flow = 37800 m^3/day (10 MGD)
Influent BOD = 157 mg/l
Maximum BOD = 235 mg/l
Effluent BOD = 20 mg/l
MLVSS = 15,000 mg/l
F/M = 0.65 day^{-1}
Flux = 9.3x10^{-3} m/sec (15 gpm/ft^2)
Recycle ratio = 3
O_2 to U-tube = 5490 kg/day (12103 lbs/day)

Reactor Characteristics

Reactor size (LxWxD) = 7.6x4.3x9.1 m (25x14x30 ft)
Number of reactors = 4
U-tube size: concentric tubes of nominal diameter 0.6 m down leg x0.9 m up leg x61.0 m deep (2ftx3ftx200ft deep)
Number of U-tubes = 4
Expected sludge production = 0.67x10^{6}kg/day (1.48x10^{6}lb/day)
Expected energy consumption = (fluidized bed and U-tube only) 4300 kwh/day
Effective sand size = 0.8 mm

The FBBT operation is expected to meet secondary discharge permit standards (BOD/SS concentrations averaging 30/30 mg/l) more cost effectively and energy efficiently than conventional activated sludge plants. These advantages are due principally to its inherent design which utilizes desirable characteristics from both trickling filter and activated sludge processes and allows higher loadings; its low volume of sludge production; and the potential elimination of secondary clarification requirements. These advantages can result in fewer and smaller treatment units performing the same degree of treatment. Preliminary indications are that overall operation and maintenance costs will remain approximately the same as activated sludge.

Innovative or Alternative Technology Characteristics

In the case of the Bay Park Wastewater Treatment Plant, expected life cycle treatment cost savings and net primary energy savings did not meet strict Innovative Technology guidelines. The construction grant eligible portion of this project excluding non-alternative sewers resulted in an estimated cost savings of 2.6% and net energy savings of 8.6% over a conventional activated sludge baseline comparison. These savings are less than the 15% and 20% cost/energy guidelines stated in the regulations.

There are some risks associated with the technology also and these were considered. In particular, more information is needed about the performance of the proprietary sand separation device and the oxygen-fed U-tube for external oxygen dissolution. These two components are considered critical to the overall performance of the system.

Notwithstanding the judgement that the proposed fluidized bed technology does not in the strictest sense meet I/A cost and energy criteria or that more operational data is needed about some of its components, it is felt the process offers substantial potential for energy and, to a lesser extent, cost savings. Pilot studies now underway will likely reduce the risk of FBBT to acceptable levels within the next six months to a year. It is further felt that the public benefit from the projected advancement in the state of the art outweighs the added risk of full-scale use. The promise of FBBT as a viable alternative to other biological treatment operations remains high.

Land Treatment of Wastewater (18)(19)

Description

Land treatment is the controlled application of liquid wastes onto the land surface by spray or surface spreading to achieve a designed degree of wastewater renovation through natural physical, chemical, and biological processes within the soil matrix.

Land treatment systems are designed to meet one or more of the following objectives: wastewater treatment as a final or intermediate process to meet regulatory limitations; wastewater disposal (zero discharge); water conservation; crop or forest growth enhancement; or landscape irrigation.

There are enough technical similarities among the various modes of land treatment to group them into three broad categories:

1. Slow Rate Systems involve the surface spreading or spray irrigation of up to 10.2 cm (4 inches) of wastewater per week onto vegetated tracts of land possessing moderately permeable soils, with the flow path being soil infiltration and percolation to groundwater. The slow rate process (also called crop irrigation) couples wastewater management with recycling of nutrients in crop production. These systems usually offer the highest degree of reliability and potential longevity.

2. Rapid Infiltration Systems constitute the percolation of large volumes of wastewater, commonly at rates measured in feet per week, through very highly permeable soils. Also known as infiltration/percolation, rapid infiltration emphasizes water reclamation rather than direct nutrient recycling.

3. Overland Flow Systems embody the controlled application of liquid wastes onto the upper portion of densely vegetated, gentle, uniform slopes possessing clay soils of very low permeability. Treatment is effected as the liquid flows slowly and evenly downslope over the soil biomat surface towards a collection system. Unlike the other land treatment systems, the product water from overland flow is almost always discharged directly to surface waters.

An interagency work group co-chaired by the USEPA and the U.S. Army Corps of Engineers prepared the Process Design Manual for Land Treatment of Municipal Wastewaters which was published in October 1977. The manual is now established as the basic resource for those who conceive, plan, and design land treatment systems in the United States. The basic unit operations and unit processes are discussed in detail and the design concepts and criteria are presented. The manual gives design examples and includes case study descriptions of operational systems.

When designing land treatment systems, it is important to recognize that no single design value or even one set of values can be realistically applied to all locations which have variable climate, geology, and treatment needs. For this reason, USEPA guidelines are varied to suit a number of possible situations and include ranges of values whenever possible. Tables 11 and 12 summarize the range of criteria for important design factors.

Land treatment is capable of achieving treatment levels comparable to the advanced wastewater treatment technologies. It achieves these levels of treatment with a comparatively low energy demand for most designs because recovery and beneficial reuse of wastewater nutrients through crop production are usually an integral part of the process. When designing land treatment systems, the health effects of wastewater contaminants in the air, on the crop, and in the groundwater must be addressed as well as soil composition

TABLE 11

COMPARISON OF DESIGN FEATURES FOR LAND TREATMENT PROCESSES

Feature	Principal Processes		
	Slow Rate	Rapid Infiltration	Overland Flow
Application techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface
Annual application rate, m (ft)	0.6 to 6.1 (2 to 20)	6.1 to 170.7 (20 to 560)	3.0 to 21.3 (10 to 70)
Field area required, 10^6 m^2 (acres)[b]	0.23 to 2.27 (56 to 560)	0.01 to 0.23 (2 to 56)	0.06 to 0.45 (16 to 110)
Typical weekly application rate, cm (in)	1.3 to 10.2 (0.5 to 4)	10.2 to 304.8 (4 to 120)	6.35 to 15.2[c] (2.5 to 6) 15.2 to 40.6[d] (6 to 16)
Minimum preapplication treatment provided in United States	Primary sedimentation[e]	Primary sedimentation	Screening and grit removal
Disposition of applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for vegetation	Required	Optional	Required

(a) Includes ridge-and-furrow and border strip.
(b) Field area in acres not including buffer area, roads, or ditches for 1 Mgal/d (43.8 L/sec) flow.
(c) Range for application of screened wastewater.
(d) Range for application of lagoon and secondary effluent.
(e) Depends on the use of the effluent and the type of crop.

TABLE 12

COMPARISON OF SITE CHARACTERISTICS FOR LAND TREATMENT PROCESSES

		Principal Processes	
Characteristics	Slow Rate	Rapid Infiltration	Overland Flow
Slope	Less than 20% on cultivated land; less than 40% on noncultivated land	Not critical; excessive slopes require much earthwork	Finish slopes 2 to 8%
Soil permeability	Moderately slow to moderately rapid	Rapid (sands, loamy sands)	Slow (clays, silts, and soils with impermeable barriers)
Depth to groundwater	2 to 3 ft (minimum) (0.61 to 0.91 m)	10 ft (lesser depths are acceptable where underdrainage is provided)	Not critical
Climatic	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often needed for cold weather

and land area requirements.

Innovative or Alternative Technology Characteristics

The Federal Water Pollution Control Act Amendments of 1972 mandated a comprehensive federal-state-local government program to reduce, prevent, and eliminate water pollution. Its intent was to redirect waste management towards recycling, reclamation, and confined disposal of wastes. The Clean Water Act of 1977 re-emphasized this intent by providing additional financial incentives for cost effective innovative and alternative approaches to waste management. It specifically included land treatment as an Alternative or Innovative Technology and required that such technologies be considered when evaluating alternative wastewater treatment operations to be partially funded by federal construction grants.

USEPA policy and guidance specific to land treatment has included two major actions. The design manual discussed earlier provided a major source of technical information and guidance in land treatment facility planning. A major EPA policy statement issued October 1977 promised vigorous support of land treatment and a critical assessment of reasons given for rejecting land treatment or requirement for high levels of preapplication treatment. These actions, in addition to the financial incentives for innovative and alternative technologies provided by the Clean Water Act, are intended to encourage major progress towards achieving the recycling, reclamation, and reuse intent of the 1972 and 1977 Acts on control of water pollution.

Municipal Wastewater Aquaculture (7)(20)

Description

Aquaculture is the production of aquatic organisms under controlled conditions. Aquatic flora and fauna are included for both marine and freshwater systems. Historically, aquaculture has been practiced for centuries for the production of food, fiber, and fertilizer. The application of waste materials to produce aquaculture systems for their fertilizer value is a more recent practice in some parts of the world. Only during the past ten years however has the use of designed aquaculture systems for treatment and management of municipal wastewaters been given serious consideration.

Aquaculture is most often considered for nutrient removal and polishing treated effluents. Studies investigating wastewater treatment using these natural systems have focused on three principal areas:

1. The culture of water hyacinths (Eichhornia crassipes) for treatment of municipal wastewater has progressed more rapidly than those utilizing other plants. A native of South America, water hyacinths are found naturally in waterways, bayous, and other backwaters of the U.S. Insects and disease have little effect on water hyacinths and they thrive in raw and partially treated wastewater. Wastewater treatment is accomplished by passing it through a hyacinth-covered basin. Batch treatment and flow through systems with single or multiple cells are used. The plants remove nutrients, BOD, suspended solids, and heavy metals.

2. Natural wetlands such as marshes, bogs, swamps, both marine and freshwater, have inadvertently served man as natural waste treatment systems for centuries. A wide diversity of plant species allows wetland vegetation to survive added stresses of concentrated wastewater effluents. Constructed artificial wetlands can be designed to meet specific project conditions while providing new wetland areas that improve wildlife habitats. Wastewater treatment in natural or artificial wetland systems is generally accomplished by sprinkling or flood irrigating the wastewater into the wetland area, or by passing the wastewater through a system of ponds, channels, or basins where aquatic vegetation is actively growing. Managed wetlands have been shown to reliably provide pH neutralization and some reduction of nutrients, heavy metals, organics, BOD, COD, SS, fecal coliforms and pathogenic bacteria.

3. Three approaches have been explored for the culture of fish utilizing municipal wastewater. These are fish stocking directly into sewage lagoons; addition of sewage effluent to

fish ponds; and conversion of sewage into lower food chain organisms such as algae and micro-invertebrates before discharging into food ponds. The most popular fish investigated have been from the carp family. A very real problem with the production of fish exposed to wastewater is ultimate use of the fish. The risks associated with human consumption of these fish are not quantified with any degree of confidence, and other uses such as fertilizer or feed material may not be cost effective.

Design criteria is very site specific. Like land treatment, no single design value or even one set of values can realistically be applied to all locations which have variable climate, geology, and treatment needs. Climate is a major limitation since effective treatment is linked to the active growth phase of the emergent vegetation. Ranges of design and performance data reported for aquaculture systems using water hyacinths or artificial wetlands receiving secondary effluent or higher quality wastewater are summarized in Table 13.

TABLE 13

RANGES OF DESIGN AND PERFORMANCE DATA FOR SOME AQUACULTURE SYSTEMS (7)*

Design Ranges	Performance Ranges	
Detention Time 4-15 days	BOD Reduction:	35-95%
Land Requirements	COD Reduction:	43-87%
2.1-16.1 m^2-day m^3 (2-15 acres/MGD)	TSS Reduction:	29-87%
Depth 0.6-1.5 m (2-5 feet)	N Reduction:	40-94%
	P Reduction:	0-94%
	Heavy Metals:	highly variable

*Aquatic plants receiving secondary or better wastewater quality.

Subject to temperature constraints, municipal wastewater aquaculture appears reliable from a mechanical and process standpoint. It has the potential for development as an economical, simple, and effective alternative for municipal wastewater treatment and management. It can yield a product of economic value (such as fertilizer) and there are indications that some systems can be energy producers by conversion of a crop such as water hyacinths, to a fuel. Information available at this time is not adequate to consider it fully developed technology for design of operational aquaculture systems treating municipal wastewater inputs. More data acquisition and evaluation is needed from different aquaculture systems to obtain reliable design criteria. Proceedings from a USEPA seminar entitled, "Aquaculture Systems for Wastewater Treatment," held at the University of California at Davis, September 11-12, 1979, are soon (at this writing) to be published and should provide additional information about aquaculture system performance.

Innovative or Alternative Characteristics

Aquaculture is specifically mentioned, along with agriculture and silviculture, in the Federal Water Pollution Control Act Amendments of 1972 as an alternate wastewater treatment method for use in meeting national water quality goals. The Clean Water Act of 1977 further encourages the consideration of Alternative Technologies such as aquaculture by providing additional construction cost assistance when they can be shown to be reasonably cost effective compared to more conventional methods of treatment. Aquaculture is a natural, potentially revenue-producing alternative to conventional wastewater treatment methods and can contribute to environmental enhancement as well as remove water contaminants.

Combined Sludge/Solid Waste Disposal and Energy Recovery System (21)(22)

Description

The disposal/energy recovery system presently under design by the City of Memphis, Tennessee will convert sorted and shredded solid waste (referred to as Refuse Derived Fuel--RDF), industrial liquids, and municipal sludge to steam at a sludge/waste energy conversion center (SWEC) using multi-hearth incineration furnaces. Revenue is gained from the steam sold to private industry (95%). Additional revenue is made selling ferrous metals separated at the RDF facility (5%). Reject solid waste materials go to a Tennessee approved landfill.

RDF is used in lieu of fuel oil in the multi-hearth furnaces to incinerate dewatered sludge from the North and South (Maxson) Wastewater Treatment Plants. Coal/RDF waterfall incinerators are provided for back-up and peak demand periods to insure steam production levels.

Sludge comes from the nominal 302,400 m^3/day (80 mgd) South and 510,300 m^3/day (135 mgd) North wastewater treatment plants. Influent to these plants is high in organic matter (BOD $\geq$400 mg/l). Sludge treatment facilities will be upgraded at both locations to provide a sludge cake of sufficient quality for incineration. The industrial liquid waste contributes only 37.9 m^3/day (10,000 gpd) to the SWEC. Sludge/solid waste process flow is outlined in Figure 10.

Design of the multi-hearth furnace had two major governing criteria. The first was that proper temperature and excess air requirements be met in the afterburner to insure burn out of the hydrocarbons so that air pollution standards would be met. Secondly, it was designed to perform satisfactorily, i.e., overcome control problems, in view of the fact that the quality (composition heating value) of its two major fuels, RDF and sludge, would vary.

In the design, values of selected characteristics of the sludge and RDF were determined as reasonable. Sludge quantities to be incinerated were expected to be 907,185 kg/day (1,000 tons per day). RDF quantities to do this were then estimated and it was determined there would be enough RDF

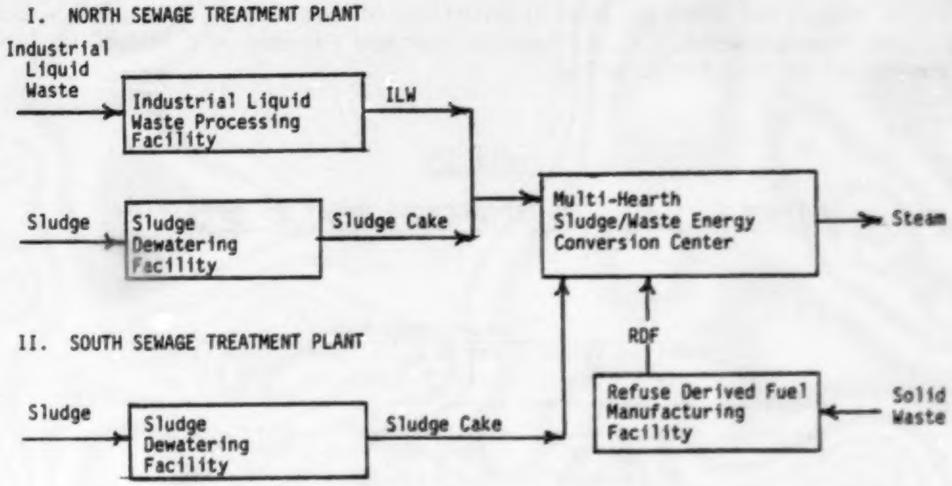

I. NORTH SEWAGE TREATMENT PLANT
Industrial Liquid Waste
Industrial Liquid Waste Processing Facility
ILW
Sludge
Sludge Dewatering Facility
Sludge Cake
Multi-Hearth Sludge/Waste Energy Conversion Center
Steam
RDF
II. SOUTH SEWAGE TREATMENT PLANT
Sludge
Sludge Dewatering Facility
Sludge Cake
Refuse Derived Fuel Manufacturing Facility
Solid Waste

FIGURE 10

COMBINED SLUDGE/SOLID WASTE DISPOSAL AND ENERGY RECOVERY FACILITIES

produced to incinerate the sludge. Basic design criteria and assumptions for the multi-hearth designs were as follows:

Multi-Hearth Afterburner Requirements:

816°C (1500°F) @ 100% excess air (or equivalent)

Sludge Characteristics:

Quantity to be Incinerated: 907,185 kg/day (1,000 TPD)
Composition: 80% Water, 11% Volatiles, 9% Ash
Gross Heating Value: 0.71 KwH/kg (1,100 BTU/lb)
Sludge Volatile Analysis / Combustion Product Analysis } Air Pollution Considerations

RDF Characteristics:

Composition: 20% Water, 70% Volatiles, 10% Ash
Gross Heating Value: 4.52 KwH/kg (7,000 BTU/lb)
RDF Volatile Analysis / Comubstion Product Analysis } Air Pollution Considerations

Final design of the system required a 1:1 ratio of sludge to RDF by weight for adequate incineration. As noted previously, it was recognized that fuel quality would vary (via percent moisture and, therefore, heating value). The multi-hearth furnace was designed to operate within a range of conditions requiring more or less quantities of fuel to maintain afterburner temperature requirements. Multi-hearth furnace recommended modes of operation are summarized in the table below.

TABLE 14

COMBINED FUEL FURNACE RECOMMENDED MODES OF OPERATION

	Operational Mode		
Fuel	Optimum Values (Case A) A1/A3	Design Basis (Case A) A2	Sub-Optimum Values (Case B) B1/B3
Wet Sludge	0.91×10^6kg/day (1000 TPD) @0.710 kwh/kg (1100 BTU/lb) (80% H_2O)	0.91×10^6kg/day (1000 TPD) @0.710 kwh/kg (1100 BTU/lb) (80% H_2O)	0.91×10^6kg/day (1000 TPD) @0.568 kwh/kg (880 BTU/lb) (84% H_2O)
RDF	0.67/0.61 kg/day (740/674 TPD) @4.518 kwh/kg (7000 BTU/lb) (20% H_2O)	0.91×10^6kg/day (1000 TPD) @4.518 kwh/kg (7000 BTU/lb) (20% H_2O)	1.16/1.04kg/day (1278/1150) @3.550 kwh/kg (5500 BTU/lb) (30% H_2O)

Innovative or Alternative Technology Characteristics

The desirability of the multi-hearth furnace was based on the difference between the cost of auxiliary fuel (RDF) and the revenue generated from the live steam. This was done by considering the sludge/RDF weight mixture, appropriate heat values, excess air, and incineration requirements. A heat balance or combustion calculation exercise was done for each operational mode and total steam generation made. Cost of the RDF fuel supplied was subtracted from the value of the steam. A "net gain" (or loss) was found. The net gains for the operating conditions defined previously are:

Net Gain ($/Day)

A_1	A_2	A_3	B_1	B_3
5,259	7,076	4,168	6,227	4,654

Some brief notes about these figures: Coal was examined as an alternate for RDF and not explored further because it had less steam-producing value and less net gain or revenue was produced. The cost of RDF fuel was taken as $\$3.50/10^6$ BTU produced. Steam value was estimated at $2.00/1,000 lb sold.

The expected net gain or net energy output exceeded by over 20% that energy which would have been required by a single purpose sludge-only project and, therefore, the co-disposal project was considered Innovative.

Because the project involves co-disposal of solid and liquid wastes, only the portion which involves sludge disposal is considered eligible for federal construction grant funds. The eligible portion was found by considering what fraction of the entire project a sludge-only disposal project would be then increasing it by the 15% incentive which interim USEPA policy allows for Alternative or Innovative projects.

$$\text{Grant Eligible Fraction} = 1.15\,\frac{(\text{Present Worth of Single Purpose Sludge-Only Project})}{(\text{Present Worth of Multiple Purpose Project Minus Present Worth of Energy Required by Sludge Treatment and Disposal Facilities})}$$

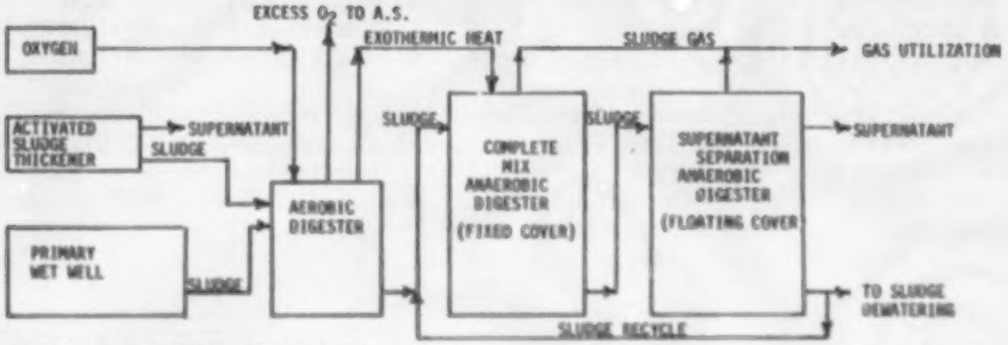
Cost of the co-disposal project was multiplied by this fraction and funded as Innovative Technology at 85%. Since approval of this project, USEPA policy has changed regarding eligible portions of co-disposal projects. The eligibilities must be determined on a case-by-case basis using present policy.

Combined Aerobic/Anaerobic Sludge Digestion Process (23)(24)

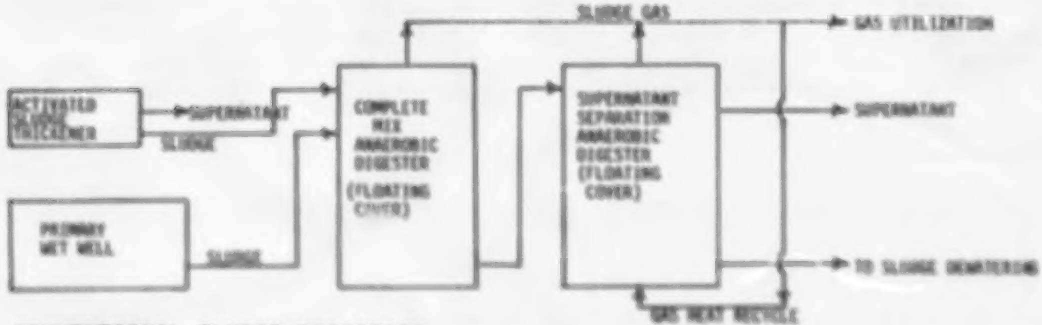
Description

The City of Lackawanna, New York is proposing to replace existing anaerobic sludge digestion facilities at their 17,010 m^3/day (4.5 mgd) oxygen activated sludge wastewater treatment plant by a combined aerobic/anaerobic digestion process. The combined digestion process (CDP) receives heat necessary for anaerobic digestion under mesophilic or thermophilic conditions from a preceding oxygen-fed aerobic digestion step which oxidizes part of the volatile suspended solids (VSS). The VSS destroyed during aerobic digestion include unionized volatile acids and total volatile solids which, if in high enough concentrations, can cause upsets in anaerobic digesters.

Figure 11 is a schematic of the CDP process and a conventional activated sludge process for comparison. Sludge from primary clarifiers and thickened waste activated sludge from a pure oxygen secondary process is to be treated. An alum sludge will also be present. The combined sludges will be first treated in an aerobic tank with a retention time of 1.2 days which generates enough heat for either mesophilic or thermophilic anaerobic digestion in a second step. In the aerobic digester, pure oxygen will be used to minimize heat losses through the vent gas. The vent gas, which will contain unused oxygen, will be recycled in the secondary treatment process. Additional oxygen-generating capacity will be required as the amount of additional oxygen required in Step 1 of the sludge digestion process exceeds the spare capacity of the secondary treatment oxygen-generating units. Dissolved oxygen in the aerobic digester is estimated to be 2 mg/l. Enough heat will be generated by the process to maintain a 54°C (129°F) temperature.


EXCESS O2 TO A.S.
OXYGEN
EXOTHERMIC HEAT
SLUDGE GAS
GAS UTILIZATION
ACTIVATED SLUDGE THICKENER
SUPERNATANT
SLUDGE
SLUDGE
SLUDGE
COMPLETE MIX ANAEROBIC DIGESTER (FIXED COVER)
SUPERNATANT SEPARATION ANAEROBIC DIGESTER (FLOATING COVER)
SUPERNATANT
AEROBIC DIGESTER
PRIMARY WET WELL
SLUDGE
TO SLUDGE DEWATERING
SLUDGE RECYCLE

COMBINED SLUDGE DIGESTION


SLUDGE GAS
GAS UTILIZATION
ACTIVATED SLUDGE THICKENER
SUPERNATANT
SLUDGE
COMPLETE MIX ANAEROBIC DIGESTER (FLOATING COVER)
SUPERNATANT SEPARATION ANAEROBIC DIGESTER (FLOATING COVER)
SUPERNATANT
PRIMARY WET WELL
SLUDGE
TO SLUDGE DEWATERING
GAS HEAT RECYCLE

CONVENTIONAL SLUDGE DIGESTION

FIGURE 11

COMBINED AND CONVENTIONAL TWO STAGE ANAEROBIC DIGESTION

The Step 2 anaerobic digestion will take place in the two existing 15.2 m (50 feet) diameter tanks which will be remodeled. Within one tank will be installed a new 9.1 m (30 feet) diameter fixed cover anaerobic digester with gas mixing. The temperature is estimated to be 50°C (122°F). Digested sludge from the first stage anaerobic tank will be fed to a second tank to be used for supernatant separation and additional gas collection. Digested sludge from the anaerobic second stage can be recycled to the first stage to insure sufficiently high SRT. A floating cover is to be provided. The gas spaces between the two anaerobic tanks will be interconnected. Total retention time in the anaerobic digesters is eight days. Sludge will be dewatered by centrifugation with existing drying beds as standby. Supernatant and centrate will be treated to remove phosphorus. Sludge disposal is proposed to be by contract. The contractor will dispose of the sludge in a landfill.

In the dual digestion process, auxiliary heating of the anaerobic digester is unnecessary because of exothermic first-stage oxidation due to closed tank use of pure O_2. This allows utilization of digester gas energy for other than anaerobic digester heating purposes. Out of 161,310 m^3 of digester gas expected every year, 107,540 m^3 can be used for in-plant heating and 53,770 m^3 will be left for power generation. The actual power demand to be satisfied is 51 kw. The comfort heating boiler and generating equipment will be arranged so that heating will take first priority and power generation will take surplus only.

The combined aerobic/anaerobic digestion seeks to incorporate advantages of each type of digestion and minimize their drawbacks. Aerobic digestion is inherently a more stable and quicker oxidation process, but it consumes more energy. Anaerobic digestion, although a slow process and susceptible to upset, has a low energy requirement and produces methane. Using the processes in sequence allows the aerobic reactor (oxygen fed) to provide adequate heat for anaerobic digestion, which in turn frees the methane produced for other energy uses. Table 15 summarizes the design parameters for the Lackawanna CSD process.

TABLE 15

PRELIMINARY DESIGN DATA FOR LACKAWANNA SLUDGE DIGESTION

Raw Sludge	
Primary:	1361 kg/day (3,000 lbs/day)
Secondary:	1179 kg/day (2,600 lbs/day inclusive of 800 lbs/day alum sludge)
Combined:	2540 kg/day (5,600 lbs/day)
Flow:	60.5 m^3/day (16,000 gpd)
Solids Content:	4.2% TSS
VSS/TSS:	0.63
Temperature:	Minimum 6°C (43°F)
Operating Conditions	
Aerobic Digester:	
Retention Time:	1.2 days
Dimensions:	[illegible]
Temperature:	54°C (129°F)
Oxygen Consumption:	1.0 kg/kg VSS destroyed in Step 1
Oxygen Utilization:	95 percent
D.O.:	2 mg/l
Heat Value of VSS Destroyed in Step 1:	10,000 BTU/lb
Anaerobic Digester:	
Retention Time:	8 days
Dimensions:	[illegible]
Temperature:	50°C (122°F)
VSS Destruction:	43 percent
Digester Gas Destroyed in Anaerobic Stage:	0.81 kg/m^3 VSS (13ft³/lbVSS)

Innovative or Alternative Technology Characteristics

Utilization of aerobic followed by anaerobic sludge digestion allows potentially more reliable and stable sludge treatment operation. As indicated previously, aerobic digestion is less subject to upset because aerobic bacteria are faster growing. Aerobic digestion converts a highly biodegradable fraction of the sludge to simpler cell matter and heat, in effect providing a more uniform, easier to digest feed to the more sensitive anaerobic process. Furthermore, because no nitrification occurs in the aerobic digester (the temperature is too high), ammonium bicarbonate is formed which has a high buffering capacity against low pH upsets. The overall result is a more rapid and extensive solubilization of the substrate.

A unique aspect of the CSD process is the possibility to pasteurize the sludge because of the relatively high temperatures ($> 52^{o}C$) and detention times encountered in thermophilic aerobic digestion. If properly stabilized and excessive heavy metals are not present, the sludge can be disposed on land.

The CSD at Lackawanna is energy efficient. All of the methane gas generated by the anaerobic digester is recovered and productively utilized. Thus, by definition, the digestion qualifies as Alternative Technology. Unlike conventional digestion, heat required for digestion is supplied from the preceeding aerobic digester, freeing the heat generated as methane gas for other purposes. The amount of oxygen needed for aerobic digestion (to proceed exothermically) exceeds the extra capacity of the pressure swing absorption (PSA) oxygen-generating facilities at Lackawanna used for the secondary, pure oxygen activated sludge treatment. Additional capacity has to be provided, but the energy required for the additional requirements are more than offset by the energy contained in the methane gas.

An energy balance around the digestion process indicated that conventional two-stage anaerobic digestion would yield from 1.08 to 1.62×10^{6} KwH per year. Digester heating requirements are estimated at 1.35×10^{6} KwH/year and power requirements are estimated at $.06 \times 10^{6}$ KwH/year. Therefore, the net annual energy yield is -0.33 to 0.21×10^{6} KwH/year. During the cold months, supplemental heating would be required.

The CSD process is estimated to yield about 0.16×10^{6} m^{3}/year digester gas or 1.00×10^{6} KwH/year. Mechanical energy for additional oxygen generation and mixing are estimated to be 0.42×10^{6} KwH/year. The net energy yield is, therefore, 0.58×10^{6} KwH/year as compared with -0.33 to 0.21×10^{6} KwH/year for conventional anaerobic digestion.

Since this is a newly-developed process, there is some risk that the benefits claimed will not be realized at full scale. Results are available from bench scale testing (14 l reactors) and pilot scale testing (to 1,516 l reactors) at Tonawanda, New York. There is an on-going demonstration project at Hagerstown, Maryland. Because it has successfully progressed beyond the bench scale and small pilot plant stage and has been adequately tested in a successful field demonstration project, it does meet USEPA requirements for developed technology. Preliminary cost data also indicate that the CSD process is cost effective compared to conventional anaerobic digestion for use at the Lackawanna Wastewater Treatment Plant.

In summary, the CSD process has adequately demonstrated the potential for increased operational stability, additional environmental benefits, and possible cost and/or energy savings over conventional anaerobic digestion. For these reasons it is considered an Innovative Technology.

V. PROJECTED IMPACT AND BENEFITS OF THE INNOVATIVE AND ALTERNATIVE TECHNOLOGY PROGRAM (25)

The foregoing discussion has centered on the recent history of design practice and construction of publicly owned treatment plants in the United States including the Innovative and Alternative Technology Program initiated to implement the new goals of the Clean Water Act of 1977. Although it is still very early in the program to draw definitive conclusions, it is clear that this program is already beginning to impact design practice. The more immediately obvious effects are an increase in cost effectiveness analysis efforts, inclusion of a comprehensive energy study as a part of facility planning, the consideration of a much broader number of alternatives and, finally, consideration of higher risk/higher benefit technologies.

Both Congress and the USEPA are vitally interested in measuring the progress of the program at the earliest possible stage in order to formulate recommendations for extending authorizing legislation and revision or fine tuning of implementing regulations.

In an effort to estimate the overall effectiveness of the Innovative and Alternative Technology Program, USEPA's Office of Research and Development completed an analysis of program impacts in February of 1980. This analysis was based on experience gained through the first 1 1/2 years of the initial three-year program. The principal data bases used in the analysis were all the Innovative and Alternative Technology project facility plans submitted to USEPA; historical USEPA Construction Grant records, the 1978 National Municipal Technology Needs Survey, consultation with state and federal review authorities; and the experience of the Office of Research and Development regarding the applicability of emerging new technologies. The analysis was further based on the assumption that the Innovative and Alternative technology set-aside funds for fiscal years 1979, 1980, and 1981 for both Innovative and Alternative Technologies would be obligated and that current regulations and qualifying criteria, as described in the Innovative and Alternative Technology Assessment Manual (7) would be followed. The analysis considered the impacts for three time periods: the initial three-year period--fiscal years 1979, 1980, and 1981; a one-year extension for fiscal year 1982; and a four-year extension for fiscal years 1983 through 1986.

For fiscal years 1979, 1980, and 1981, 17% of construction grant projects are estimated to qualify as Innovative Technologies. For fiscal years 1979 and 1980, 25.5% of the projects are estimated to qualify for Alternative Technologies. This will increase to 42.5% in fiscal year 1981. Collectively, 42.5% of the construction grant projects are estimated to qualify for Innovative and Alternative Technologies for fiscal years 1979 and 1980, and 59.5% for fiscal year 1981.

In fiscal year 1979, an expenditure of $21 million of Innovative set-aside funds, which is equivalent to $716 million of construction grant funds, is estimated to save $133 million due to adoption of Innovative Technology. This is equivalent to a return of 632% for each Innovative set-aside dollar spent, or a saving of 18% based on the total construction grant dollars expended. An expenditure of $63 million of fiscal year 1979 Alternative set-aside funds, which is equivalent to $1.07 billion of construction grant funds, is estimated to save a total of $365 million due to the use of Alternative Technology. This is equivalent to a return of 579% for each Alternative set-aside dollar spent or a saving of 34% based on construction grant dollars experded. In summary, a total of $84 million spent as Innovative and Alternative set-aside funds, which is equivalent to $1.786 billion of construction grant funds, will result in $498 million or 28% total construction grant savings. Savings resulting from the Innovative and Alternative Technology Program for fiscal years 1980 and 1981 have also been estimated and shown in Table 16.

TABLE 16

SUMMARY OF PROJECTED COST SAVINGS FROM THE NATIONAL INNOVATIVE AND ALTERNATIVE TECHNOLOGY PROGRAM

Fiscal Year	I/A Set-Aside ($Million)			Innovative		Alternative	
	I	A	I+A	Saving ($ Million)	$ Saved / $ Set-Aside Spent	Saving ($ Million)	$ Saved / $ Set-Aside Spent
79	21	63	84	133	6.32	365	5.79
80	17	51	68	106	6.32	295	5.79
81	17	85	102	106	6.32	492	5.79
82	22.5	112.5	135	142	6.32	652	5.79

The total construction grant appropriations, annual setaside funds, projected number of Innovative and Alternative projects, perentage of each project identified as Innovative or Alternative Technology, nd the dollar savings resulting from each is summarized in Table 17.

In terms of dollars saved per dollar expended, the Innovative and Alternative portions of the program are about equal, with the forer slightly more effective than the latter. In the Innovative Technologes, the 15% life cycle cost criteria is about ten times more effective i savings than the 20% energy saving criteria. In the Alternative Technolgies, savings from sludge handling and associated energy conservation and ecovery systems are insignificant due to their low projected activity volumeand low savings margin. About two-thirds of the savings in the Alternative chnologies are

TABLE 17

NATIONAL IMPACTS OF THE INNOVATIVE AND ALTERNATIVE TECHNOLOGY PROGRAM ON THE CONSTRUCTION GRANTS PROGRAM

Fiscal Year	Grant Appropriation ($ Billion)	Total(1) Appropriation For Step 3 Projects ($ Billion)	Innovative and Alternative Set Aside ($ Million) I	A	Total(2) No. of Grant Projects Step 3	No. of(3) Innovative Projects	No. of(4) Alternative Projects	Total No. of I + A Projects	Dollar Value of Innovative(5) Projects ($ Million)	Portion of(6) Projects Identified As Innovative(%)	Dollar Value(7) of Alternative Projects ($ Million)	Portion of(8) Projects Identified As Alternative(%)	Dollar Value(9) of I + A Projects ($ Million)	Portion of(10) Projects Identified As I + A (%)
79	4.2	3.91	21	63	1,005	184	275	459	716	17	1,070	25.5	1,786	42.5
80	[illegible]	[illegible]	17	51	867	148	223	371	576	17	867	25.5	1,443	42.5
81	[illegible]	[illegible]	17	85	867	148	371	519	576	17	1,443	42.5	2,019	59.5
82	[illegible]	[illegible]	22.5	112.5	1,079	197	492	689	766	17	1,914	42.5	2,680	59.5

NOTES: (1) Based on Step 3/(Step 1 + Step 2 + Step 3) = 93% Also see Exhibit I
(2) Based on an average project federal share of $3.89 million. For FY 79, $3.91 billion/$3.89 million = 1,005 Also see Exhibit I
(3) Based on 1/2% set-aside for innovative projects. For FY 79, $21 million x 8.5 x 4/$3.89 million = 184 Also see Exhibit II
(4) Based on 1 1/2% set-aside for 1979 and 1980, and 2 1/2% set-aside for 1981 and 1982 for alternative projects. For FY 79, $63 million x 8.5 x 2/$3.89 million = 275. For FY 81, $85 million x 8.5 x 2/$3.89 million = 371 Also see Exhibit II
(5) For FY 79, 184 x $3.89 million = $716 million
(6) For FY 79, $716 million/$4.2 billion = 17%
(7) For FY 79, 275 x $3.89 million = $1,070 million
(8) For FY 79, $1,070 million/$4.2 billion = 25.5%
(9) For FY 79, $716 million + $1,070 million = $1,786 million
(10) For FY 79, $1,786 million/$4.2 billion = 42.5%

TABLE 17 (Continued)

EXHIBIT I

Calculations of Average Project Cost and Average Project Federal Share:

From OWPO Fact Sheet, as of October 31, 1979, grants awarded for Step 3 are 21.6 billion for 5,554 grants.

21,600,000,000/5,554 = $3,889,089/grant or about $3.89 million federal share per project

3.89/0.75 = $5.19 million project cost per project

Assume the above costs are based on second quarter of 1977 money as being used in MCD-37.

From MCD-37, $5.19 million project cost is equivalent to a construction cost of $4.23 million ($5.19 million/1.2264=$4.23 million), which is for a secondary treatment, new construction with a flow rate of 2.3 mgd. Use 2.5 mgd as the average plant size.

Step 3 grants/(Step 1 + Step 2 + Step 3) = $21.6 billion/$23.2 billion
= 93%

For FY79, Appropriation for Step 3 = $4.2 billion x 0.93
= $3.91 billion

Total number of grant projects = $3.91 billion/$3.89 million
= 1,005

EXHIBIT II

Calculations of Innovative and Alternative Projects, Number of Projects and Federal Share

Assume ratio of innovative (I) projects to alternative (A) projects is 1:3 for FY79 and 80, and 1:5 for FY81 and 82 based on quarterly report of 40 I to 216 A. 25% of all I projects qualify for 85% funding, and 50% of all A projects qualify for 85% funding. (Exhibit X).

For FY 79:

Number of Innovative Projects = $21 million/($3.89 million x 0.25/0.5)
= 184

Number of Alternative Projects = $83 million/($3.89 million x 0.5/0.5)
= 275

Innovative Projects Federal Share = 184 x $3.89 million
= $716 million

Alternative Projects Federal Share = 275 x $3.89 million
= $1,070 million

from land application and other natural systems because of their high activity and savings. The lower projected volume on-site and alternative collection systems contribute about one-third of the total Alternative Technology savings due to their high margins on saving which is close to twice that of land application and other natural systems.

If the Innovative and Alternative Technology Program is extended to fiscal year 1982 with 3% total Innovative and Alternative set-aside and a minimum of 1/2% set-aside for Innovative Technologies from a total construction grant appropriation of $4.5 billion, a total of $142 million is estimated to be saved by expending $22.5 million of Innovative set-aside which is equivalent to $766 million of construction grants. Savings of $652 million are projected from an Alternative set-aside of $112.5 million or construction grants of $1.914 billion. A total savings of $794 million are projected for a total Innovative and Alternative set-aside of $135 million for construction grants totaling $2.68 billion. These projected savings for fiscal year 1982 are also shown in Table 16.

The Innovative and Alternative Technology Program has the further potential to achieve significant cost and energy savings from projects to be funded in fiscal years 1982 through 1986. An estimate of this impact was developed by conducting a comprehensive review of technologies which could potentially save cost or energy or meet other goals of the Clean Water Act of 1977. A list of these technologies was compiled under seven general categories as follows: biological, chemical or physical, land application, other natural systems, sludge handling and energy recovery, energy conservation and recovery, on-site and alternative collection systems. The status of development and potential application of each technology, including applicable size ranges, treatment levels, waste streams, and geographical areas, is shown in Table 18.

Column one of Table 18 lists each of the seven general categories of technologies described previously in Table 17. Column two indicates the potential number of facilities to be funded during the four-year period which could potentially realize cost and/or energy saving benefits. These figures are estimated based upon the calculation that 3,290 construction projects would be funded according to the following analysis. Assuming that $4 billion/year will be appropriated for fiscal years 1983, 1984, 1985, 1986; $16 billion will be available for funding purposes. Out of this total, 7% would typically be used for planning and design projects according to records of the grants program from its inception. It is assumed that 13% will be collectively used for projects which would not be directly influenced by the Innovative and Alternative Technology Program. These would include such projects as infiltration/inflow and sewer replacement and rehabilitation. Therefore, 80% of the $16 billion, or $12.8 billion, remains as the amount which could be influenced directly. Using the historical average Step 3 construction cost of $3.89 million, 3,290 construction projects could potentially realize cost and/or energy savings. In column two, out of the 3,290 projects, 1,500 would include biological treatment technologies which could potentially save costs or energy. In a similar manner, 400 would include chemical or physical technologies, 1,000 would include land treatment technologies and natural systems, 1,000 would include sludge treatment and

TABLE 18

CALCULATIONS OF POTENTIAL COST AND ENERGY SAVINGS OF THE INNOVATIVE AND ALTERNATIVE TECHNOLOGY PROGRAM FOR PROJECTS TO BE FUNDED IN FISCAL YEARS 1982 THRU 1986

-1- Category of I/A Technology	-2- Potential No. of Projects Influenced by I/A Program	-3- Average Size of Baseline Conventional Facility (mgd)	-4- Calculated LCC of the Average Baseline Conv. Facility($M)	-5- Calculated Energy Use of the Average Baseline Conv. Facility, Kw hr/yr x 10^{-3}	-6- Range of LCC Savings (%)	-7- Range of Energy Savings (%)	-8- Calculated Total LCC Savings $ B (2 x 4 x 6)	-9- Calculated Total Energy Savings Kw hr/yr x 10^{10} (2 x 5 x 7)
Biological	1,500	7.5	8.65	1,125	20 - 30	20 - 30	2.6 -3.89	3.38-5.06
Chemical or Physical	400	7.5	8.65	1,125	5 - 15	5 - 15	0.17-0.52	0.23-0.68
Land Application	800	1.0	3.5	500	30 - 40	10 - 20	0.84-1.12	0.40-0.80
Other Natural Systems	200	1.0	3.5	500	30 - 40	50 - 70	0.21-0.28	0.50-0.70
Sludge Handling and Energy Recovery	1,000	7.5	8.65	1,125	10 - 20	30 - 40	0.86-1.73	3.38-4.50
Other Energy Conservation and Recovery	1,500	7.5	8.65	1,125	10 - 20	20 - 30	1.30-2.60	3.38-5.06
On-Site and Alternative Collection Systems	1,000	0.35	1.2	231	20 - 30	10 - 20	0.24-0.36	0.23-0.46
						TOTALS	6.22-10.5	11.6-17.26

associated sludge energy recovery technologies, 1,500 would include other energy saving technologies. These estimates are based upon the distribution of existing facilities, historical patterns of use for each type of technology as experienced in the Construction Grants Program and ORD experience in the Innovative and Alternative Technology Program thus far, as well as stated goals of USEPA involving, for example, the number of projects which should include land treatment and natural systems. The total number of projects in all categories is larger than 3,290 because all of the categories are not mutually exclusive.

Column three indicates the average size of a project for each category of technology which would realize cost or energy saving benefits. The 9,450 m^3/day (2.5 mgd) figure is the average size of projects historically funded by USEPA. The sizes for the other categories are smaller due to the nature of the technologies involved typically being limited to smaller communities.

Columns four and five present the calculated life cycle cost and energy use for a conventional facility not using Innovative or Alternative technologies. These values serve as a baseline for estimation of cost and energy savings. Life cycle costs used here are those present worth costs of construction and operation and maintenance costs and are calculated based on $5.19 million construction costs for a 9,450 m^3/day (2.5 mgd) facility. Approximately 60% of total life cycle costs are estimated as construction costs. Therefore, the life cycle cost for a 9,450 m^3/day (2.5 mgd) facility is $8.65 million. The life cycle cost for other size facilities was calculated using the same unit cost per mgd. For this analysis, at this size range spread, scale factors were ignored. Energy use for a conventional treatment facility was calculated using 450,000 KwH/year/mgd for a 9,450 m^3/day (2.5 mgd) facility, and 500,000 KwH/year/mgd for a 3,780 m^3/day (1.0 mgd) facility. For a 1,323 m^3/day (0.35 mgd) on-site and alternative collection system 660,000 KwH/year/mgd was used and includes an allowance for energy used for conventional collection as well as treatment. The energy figures used for treatment represent total energy requirements for a typical conventional activated sludge plant and do not include the net energy recovered from anaerobic digestion.

Columns six and seven indicate the ranges of estimated percent of life cycle cost and energy savings of innovative and alternative technologies used in lieu of conventional technologies. The percent savings are given based on total life cycle costs and total energy use in the conventional baseline facility. They are not based on unit process savings. The estimates of savings are based upon research and development experience, fundamental process considerations, and experience in the Innovative and Alternative Technology Program to date. Columns eight and nine present calculated total potential life cycle cost and energy savings for each category of technology listed.

The total present worth (capital cost plus operation and maintenance cost) dollar savings which could be potentially realized from the four-year program range from $6.22 to $10.5 billion. Of this, $3.73 to $6.3 billion would be for construction savings; and $2.49 to $4.2 billion would be for operation and maintenance savings. The construction dollar savings amount to 22% to 37% of construction dollars. User charges would be reduced due to lower operation and maintenance costs and lower local share requirements.

According to the 1978 Needs Survey, $35.6 billion is required to meet the Year 2000 needs in categories which could be directly affected by the Innovative and Alternative Technology Program. Of this, from 22% to 37% could potentially be saved amounting to $7.3-13 billion over the long term.

Potential energy savings for projects funded during fiscal years 1983 through 1986 range from 11.5 to 17.3 x 10^8 KwH/year which is 9% to 13% of the 1977 energy requirements for municipal wastewater treatment in the United States.

In addition to the immediate cost and energy impacts previously described, the benefits derived from continuation of the Innovative and Alternative Technology Program for an additional four-year period beyond fiscal year 1982 are estimated to far outweigh the additional investment. Initial program benefits are derived from: (a) demonstrating the immediate effects of a slightly higher risk as a part of design practice; (b) removal of the more obvious disincentives to innovative designs; (c) injection of arbitrary increased cost and energy saving goals to encourage improved design; and (d) strong encouragement of the increased use of certain natural treatment systems. Because of the inherent 6-8 year lag time in our planning, design, and construction process, and also because of the traditional conservative nature of consulting engineers and state review authorities and their reluctance to change, the initial three-year program will serve primarily to develop momentum in the program. Consultants and equipment manufacturers have approached the initial three-year program cautiously, questioning the benefits of learning and competing under a new set of standards and guidelines with no guarantee of further federal commitments. Because of the legislative requirements and intensive national training program, many medium and larger size consulting firms and equipment manufacturers are now equipped to aggressively pursue improved Innovative and Alternative designs (approximately 1,500 consultants have attended a series of two-day intensive Innovative and Alternative Technology seminars). Continuation of the program will maximize the investment made to develop this capability.

The Innovative Technologies proposed during the first 1 1/2 years of the three-year program were those already developed to some extent by private and public sector basic research efforts. Program continuation for an additional four years beyond 1982 is estimated to stimulate basic and developmental research needed for second generation innovative and alternative technologies. This potential is especially high in regard to improved natural systems.

The ultimate objective of the program is to persuade municipal decision makers, consulting engineers, and the general public that innovative and alternative technology performs better and is less costly than conventional approaches, without a continued need for a federal fiscal incentive program. It is anticipated that this will be accomplished by establishing as standard practice reduction of over-conservativism in design, greater consideration of natural systems, and improved cost and energy analyses.

Present experience suggests that continuation of the Innovative and Alternative Technology Program will offer the following benefits:

- Provide for improved resource recovery and beneficial use of wastewater constituents.
- Firmly establish as standard practice a reduction in over-conservative designs, a new emphasis on improved cost analysis and energy considerations, and greater consideration of alternative technologies in the design of POTWs.
- Create an incentive for improved basic research in both public and private sectors due to confidence in continuation of a market for improved technology.
- Create confidence in the consulting engineering profession and state review authorities in higher risk, higher benefit, less conservative designs.
- Increase the consideration and appropriate use of natural systems.
- Substantially increase private sector research in more reliable cost and energy efficient equipment.
- Reduce secondary energy utilization for production of chemicals and energy intensive materials of construction.
- Reduce long-term operation and maintenance costs to the community, especially future energy needs.
- Provide incentive for engineering curriculum improvements including emphasis toward natural systems and alternative technologies.

ABBREVIATIONS AND ACRONYMS

A	- Alternative Technology
atm	- atmospheres
BOD	- 5-day biochemical oxygen demand
BTU	- British Thermal Units
C	- Carbon
°C	- degrees Celsius
CDF	- combined digestion process
cm	- centimeter
COD	- chemical oxygen demand
CWA	- Clean Water Act
D	- depth
DDS	- dual digestion sludge process
$	- U.S. dollars
DWBR	- deep well biological reactor
°F	- degrees Fahrenheit
FB	- free board
FBBT	- fluidized bed biological treatment
F/M	- food to microorganism ratio
ft	- ft
FWPCA	- Federal Water Pollution Control Act
gpm	- gallons per minute
H_2	- Hydrogen
H_2O	- water
hp	- horsepower
hp-hr	- horsepower-hour
I	- Innovative Technology
I/A	- Innovative and Alternative Technology
ID	- inner diameter
in	- inch
kg	- kilogram
kw	- kilowatt
KwH	- kilowatt hour
L	- length
l	- liter
LACSD	- Los Angeles County Sanitation Districts
lb	- pound
LCC	- life cycle costs

ABBREVIATIONS AND ACRONYMS (Continued)

m - meter

MCD - USEPA Municipal Construction Division

mgd - million gallons per day

mg/l - milligram per liter

MLSS - mixed liquor suspended solids

mm - millimeter

no. - number

O_2 - Oxygen

OD - outer diameter

ORD - USEPA Office of Research and Development

% - percent

PL - Public Law

pop. - population

POTW - publicly owned treatment works

PSA - pressure swing absorption

RBC - rotating biological contactor

RDF - refuse derived fuel

sec - seconds

SS - suspended solids

SRT - sludge retention time

SWD - side wall depth

SWEC - Sludge/Waste Energy Conversion Center

TPD - tons per day

TSS - total suspended solids

U.S. - United States

USEPA - United States Environmental Protection Agency

VSS - volatile suspended solids

VWCR - Vertical Well Chemical Reactor

W - width

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PARALLEL EVALUATION OF BELT FILTER PRESSES AND LOW SPEED SCROLL CENTRIFUGES

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I. INTRODUCTION

For almost a decade the Sanitation Districts of Los Angeles County have conducted sludge dewatering research in response to increasingly stringent standards regulating effluent and sludge discharge to the ocean from the Joint Water Pollution Control Plant in Carson, California. Most recently, the sludge dewatering studies culminated with the side-by-side testing of full scale low speed scroll centrifuges and belt filter presses. A brief summary of the earlier pilot studies and the results of the full scale testing are presented in this paper.

A. EXISTING FACILITIES

The Joint Water Pollution Control Plant (JWPCP) currently provides advanced primary treatment for 15 m^3/s (350 mgd) of domestic and industrial wastewater. The plant treats approximately 75 percent of the flow in the entire Sanitation Districts system, which serves a population of four million. Other treatment plants in the system return their sludge to sewers which terminate at the JWPCP, so it serves as a central solids processing facility for the whole system.

Sludge and floatables from the primary settling tanks are anaerobically digested for approximately fifteen days and dewatered by centrifuges. Basket centrifuges are provided in sufficient capacity to dewater approximately one-half of the sludge flow. With polymer conditioning at 2 g/kg (4 lb/ton), these machines are capable of recovering over 90 percent of feed suspended solids and producing a cake with 22 percent total solids. The remainder of the sludge flow is dewatered by high speed, countercurrent flow scroll centrifuges, which are able to recover only one-third of the feed solids.

B. PURPOSE OF THE TESTS

In the 1970's, both federal and California discharge standards were enacted which required an increased level of treatment at the JWPCP, including a ban against the discharge of sludge into the Pacific Ocean. Construction is now in progress which will provide 8.8 m^3/s (200 mgd) of secondary treatment, using the high purity oxygen activated sludge process. Dewatering studies were conducted to select equipment needed for two reasons:

1. A large amount of waste activated sludge will be produced when secondary treatment commences.

2. Digested primary sludge production exceeds the capacity of the existing basket centrifuges, and the existing high speed scroll centrifuges cannot provide acceptable solids recovery for the remainder of the sludge.

II. PILOT STUDIES

Initially, a great variety of schemes were studied for the processing of waste activated sludge (WAS). These included thickening by gravity, dissolved air flotation, and centrifugation; aerobic digestion, and anaerobic digestion at both mesophilic and thermophilic temperatures; and heat treatment, as well as wet oxidation at low or intermediate pressure. WAS processed by the above methods and anaerobically digested primary sludge were dewatered by means of vacuum filter, plate and frame filter press, belt filter press, scroll centrifuge and basket centrifuge. Details of these tests have been described elsewhere.(1,2)

As a result of the pilot studies, it was decided that WAS would be thickened by dissolved air flotation, and anaerobically digested. A final decision was not made on the dewatering equipment at this time. A qualitative characterization of the dewatering performance achieved on anaerobically digested sludge is presented in Table 1. Vacuum filtration of digested waste activated sludge (DWAS) met with little success, requiring very high doses of inorganic chemicals for conditioning, and exhibiting very poor discharge of sludge from the media. Pressure filtration, although producing excellent cakes and solids recovery, also required prohibitively high dosages of conditioning chemicals. Basket centrifugation of 100 percent DWAS met with little success; however, if substantial amounts of digested primary sludge (DPS) were blended with the DWAS, acceptable performance could be achieved. The Districts' experience with full scale operation of basket centrifuges on DPS has shown this device to require the least amount of polymer conditioning to achieve good recovery, while also producing an acceptably dry cake.

The scoll centrifuge and the belt filter press were considered to demonstrate the best overall performance for both DPS and blends of DWAS plus DPS. Dewatering of 100 percent DWAS, and handling of the cake thus produced, were generally found to be less than satisfactory. These two devices were then subjected to full scale, long term testing to further evaluate operation and maintenance characteristics, as well as the conventional dewatering parameters such as cake solids, recovery, and chemical requirements.

III. DESCRIPTION OF FULL SCALE EXPERIMENTS

A. EQUIPMENT DESCRIPTIONS

Four pieces of equipment were selected which appeared to represent the best technology available at the conclusion of the pilot experiments in early 1978. Of the four machines selected, three were the largest size marketed by the respective manufacturers for this application.

TABLE 1. Dewatering Characteristics Determined by Pilot Studies

Device	Sludge Type	Cake Dryness	Chemical* Requirements	Solids Recovery
Vacuum Filter	DWAS	F	P	F
Pressure Filter	DWAS	E	F-P	E
Basket Centrifuge	DWAS	P	G	F
	DPS	G	E	G
Scroll Centrifuge	DWAS	G	G	G
	DPS	G-E	G	G-E
Belt Filter Press	DWAS	G	G	F-G
	DPS	G-E	G	G

DWAS = Waste activated sludge, anaerobically digested

DPS = Primary sludge, anaerobically digested

E = Excellent G = Good F = Fair (moderate) P = Poor

* Lime and ferric chloride used to condition sludge for vacuum and pressure filters. Polymer used for centrifuges and belt filter presses.

The four devices included:[1]

1. A 2.2 meter Winkle belt filter press (Ashbrook-Simon-Hartley Co.)
2. A 2.0 meter Magnum belt filter press (Parkson Corp.)
3. An 80 cm (32 inch) diameter Kruger 280 MC low speed scroll centrifuge (Ingersoll-Rand Corp.).
4. A 91 cm (36 inch) diameter Bird HB5900 low speed scroll centrifuge (Bird Machine Co.)

The two belt filter presses varied substantially in their design as shown in Figures 1 and 2. The notable features of the Winkle press include a very large gravity drainage zone, followed by a vertical V-shaped section which provides hydrostatic pressure to aid in dewatering and distributes the sludge evenly across the belt width; the Winkle press also employs in-line polymer mixing and flocculation rather than a separate flocculation vessel. The Magnum press design includes an inclined gravity drainage zone with relatively greater liquid depth, fabric belts operated at reduced tension to permit a less massive frame, and a third belt made of rubber strips to provide high pressure dewatering.

The two scroll centrifuges were more similar in design. Both were of the type commonly designated as "low speed," developing centrifugal force of about one thousand times gravity or less. Both could be classified as the concurrent flow type, with the sludge cake being conveyed in the same direction as the flow of influent sludge. The Kruger centrifuge is different in its use of a second scroll designed to introduce the flow to the liquid pool with less turbulence. The Kruger devotes proportionately more length to the beach or drainage area and less to the settling zone in comparison to the Bird, as can be seen in Figures 3 and 4.

It is especially notable that during the course of the testing, one of the centrifuges was eventually modified by the addition of an automatically controlled, variable speed backdrive. The backdrive of a centrifuge is the mechanism which controls the speed of the scroll conveyor relative to the bowl. This speed differential directly affects the amount of solids that are held in the bowl during operation. By automatically controlling the scroll differential in proportion to the amount of torque developed on the scroll shaft, close control of the solids retention in the bowl could be achieved, similar to the way a sludge blanket might be controlled in a sedimentation tank. The importance of this development is explained later in this paper.

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[1] Use of trade names is not intended as an endorsement of the products, but serves as a reference for design features which cannot be described within the scope of this paper.

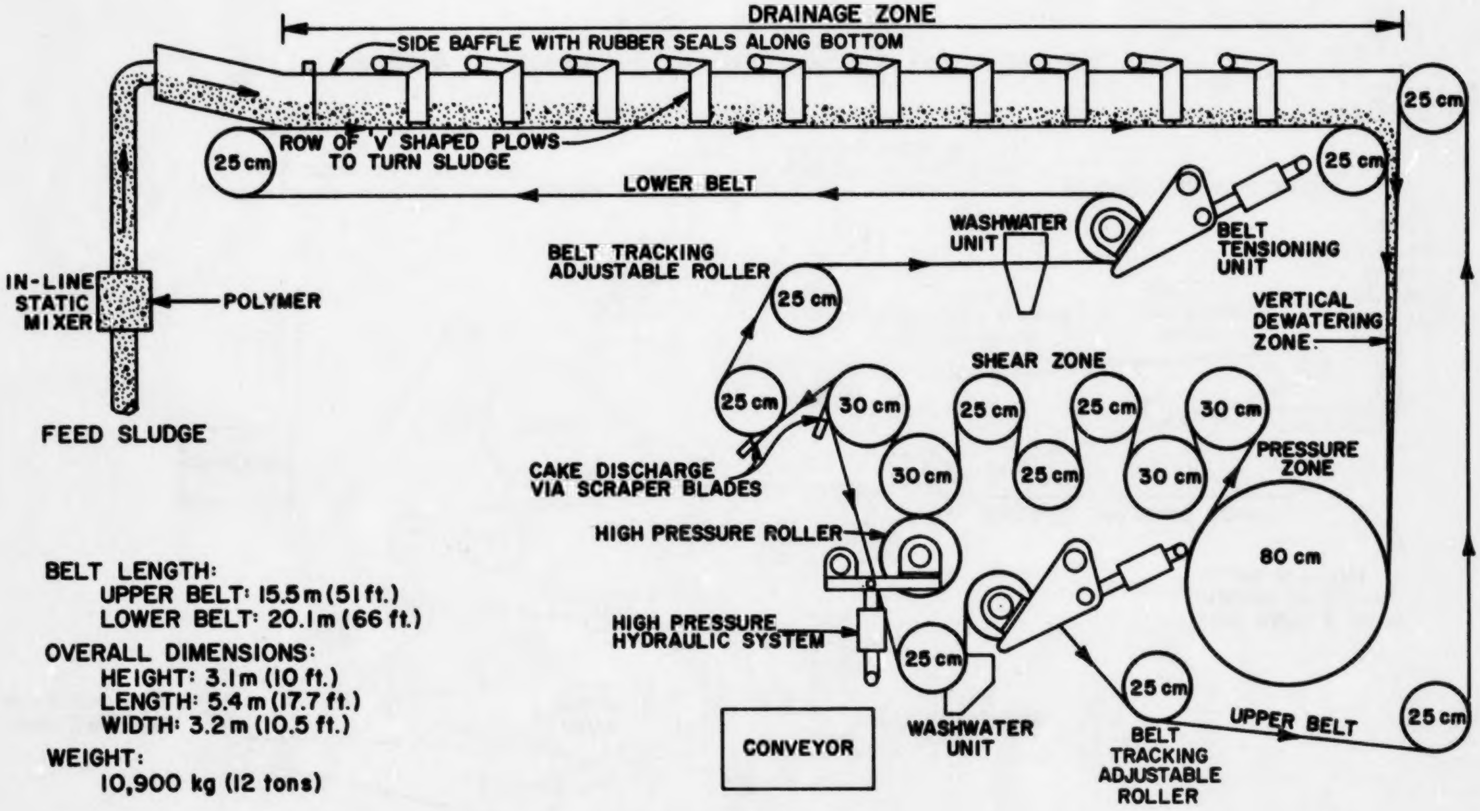
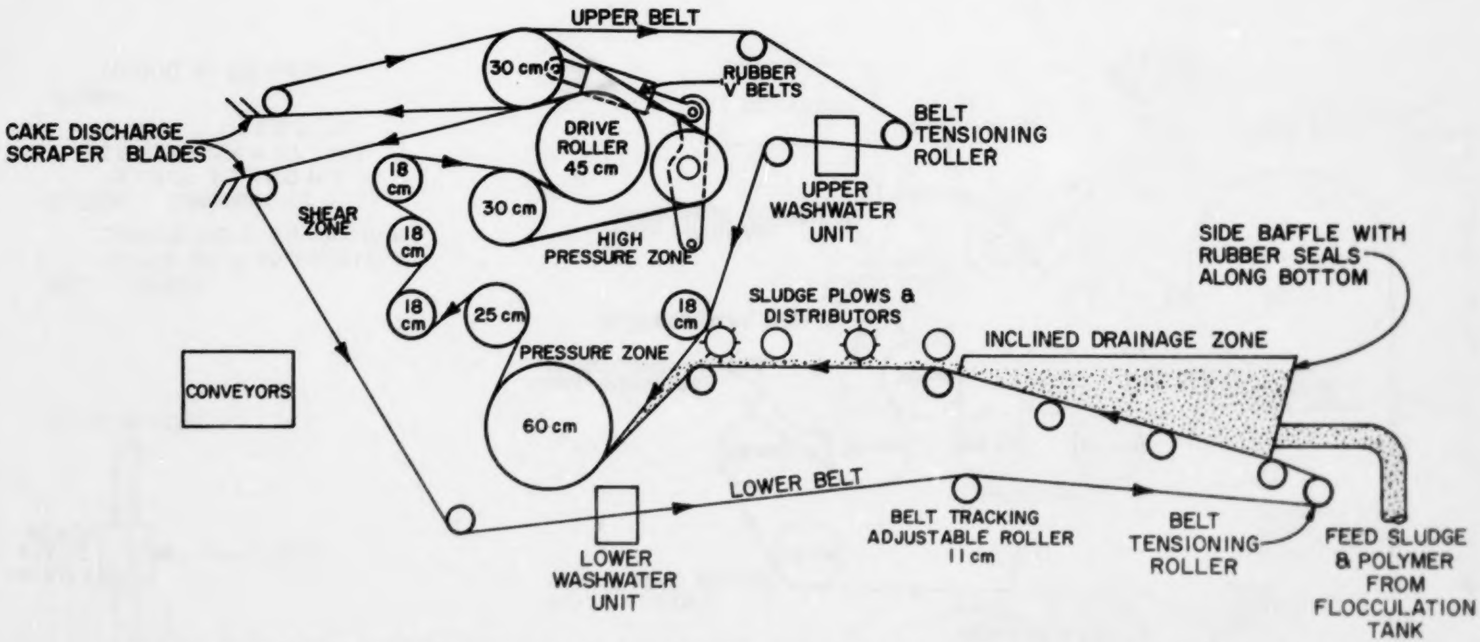

DRAINAGE ZONE
SIDE BAFFLE WITH RUBBER SEALS ALONG BOTTOM
ROW OF 'V' SHAPED PLOWS TO TURN SLUDGE
25 cm
LOWER BELT
IN-LINE STATIC MIXER
POLYMER
FEED SLUDGE
WASHWATER UNIT
BELT TRACKING ADJUSTABLE ROLLER
BELT TENSIONING UNIT
VERTICAL DEWATERING ZONE
SHEAR ZONE
30 cm
25 cm
80 cm
PRESSURE ZONE
CAKE DISCHARGE VIA SCRAPER BLADES
HIGH PRESSURE ROLLER
HIGH PRESSURE HYDRAULIC SYSTEM
CONVEYOR
WASHWATER UNIT
BELT TRACKING ADJUSTABLE ROLLER
UPPER BELT
BELT LENGTH:
UPPER BELT: 15.5 m (51 ft.)
LOWER BELT: 20.1 m (66 ft.)
OVERALL DIMENSIONS:
HEIGHT: 3.1 m (10 ft.)
LENGTH: 5.4 m (17.7 ft.)
WIDTH: 3.2 m (10.5 ft.)
WEIGHT:
10,900 kg (12 tons)

Figure 1. Schematic of the Winkle Press Model 3V.


UPPER BELT
RUBBER V BELTS
30 cm
DRIVE ROLLER 45 cm
BELT TENSIONING ROLLER
CAKE DISCHARGE SCRAPER BLADES
18 cm
18 cm
18 cm
30 cm
25 cm
18 cm
SHEAR ZONE
HIGH PRESSURE ZONE
UPPER WASHWATER UNIT
SIDE BAFFLE WITH RUBBER SEALS ALONG BOTTOM
SLUDGE PLOWS & DISTRIBUTORS
INCLINED DRAINAGE ZONE
PRESSURE ZONE
CONVEYORS
60 cm
LOWER BELT
BELT TRACKING ADJUSTABLE ROLLER 11 cm
BELT TENSIONING ROLLER
FEED SLUDGE & POLYMER FROM FLOCCULATION TANK
LOWER WASHWATER UNIT

BELT LENGTH: UPPER - 10m (32.8 ft)
LOWER - 14.1m (46.5 ft.)

OVERALL DIMENSIONS: HEIGHT - 2.3m (7.5 ft.)
LENGTH - 4.5m (14.8 ft.)
WIDTH - 2.7 m (8.7 ft.)

WEIGHT: 6820kg (7.5 tons)

Figure 2. Schematic of the Magnum Press MP 80.

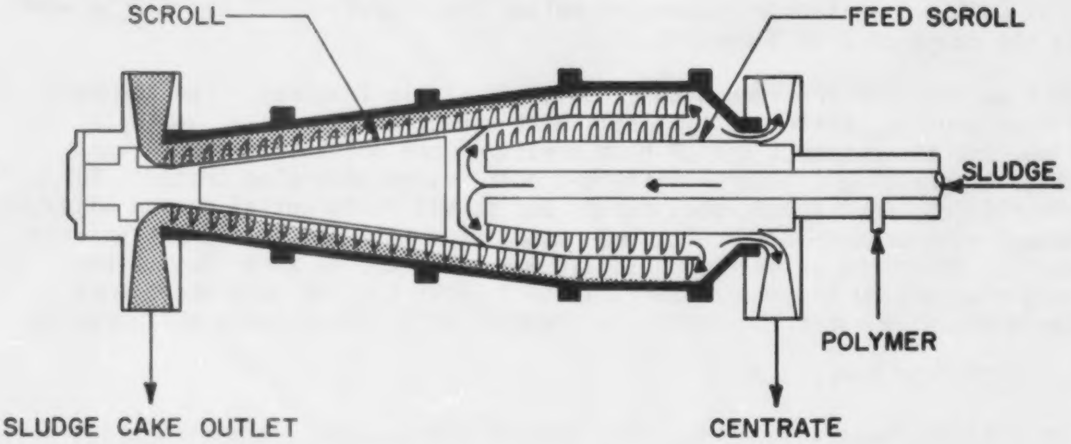

SCROLL
FEED SCROLL
SLUDGE
POLYMER
SLUDGE CAKE OUTLET
CENTRATE

Figure 3. Kruger 280MC Centrifuge.

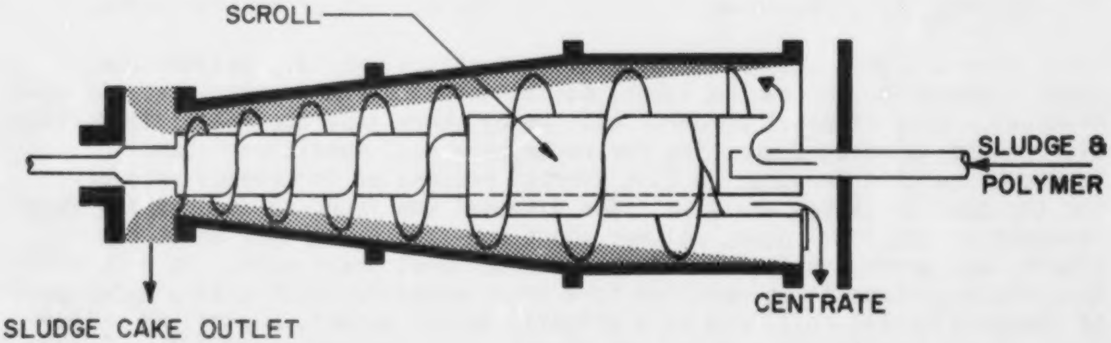

SCROLL
SLUDGE &
POLYMER
CENTRATE
SLUDGE CAKE OUTLET

Figure 4. Bird HB5900 Centrifuge.

B. EXPERIMENTAL VARIABLES

Throughout the testing of the four devices, cake dryness and suspended solids recovery were the dependent variables measured, while sludge feed rate and polymer dosage were controlled as independent variables. The feed sludge used for most tests was DPS with a suspended solids concentration of 2.5 to 3.0 percent. Various blends of DPS and DWAS were also tested to the extent that DWAS was available; suspended solids concentrations of the blends were in the range of 2 to 3 percent.

Each machine was provided with certain adjustable features. For the belt filter presses, these are typically belt speed, belt tension, washwater flow, and the optional use of high pressure zone mechanisms in the last stage of pressing. Several different belt weaves were also tested. For the centrifuges, bowl speed, pool depth, and scroll differential can be adjusted, though with greater difficulty than associated with the belt press adjustments. After the automatic backdrive was added to the centrifuge, scroll torque became an important new variable. These machine parameters were systematically varied to determine their effects and optimize performance.

C. METHOD OF EVALUATION

The ultimate objective of the test program was to select a system which would result in the lowest total cost to dewater and dispose or reuse the sludge generated at JWPCP. Therefore, while the test program was being conducted, the results were continually subjected to an economic analysis that reflected the total system cost for sludge processing. This provided the means to evaluate the relative importance of cake dryness, polymer dosage, and hydraulic capacity, thus providing direction for the conduct of further tests.

Total cost included the following items: amortized capital; polymer use; power consumption; operating labor; maintenance labor and materials; and cake disposal. Cake disposal at JWPCP will be by three separate means: landfilling with refuse; windrow composting for reuse as a soil conditioner; and Carver-Greenfield dehydration plus thermal processing for energy recovery. For the typical system analysis, cake disposal was found to account for about one-half of the total cost, polymer about one-fourth, capital about one-eighth, and power, operation and maintenance about one-eighth. It is clear that these circumstances resulted in a great emphasis on obtaining a dry cake to reduce disposal cost, and to a slightly lesser extent, minimizing polymer use. Flow capacity per machine and direct power consumption for dewatering were found to influence total costs to a much smaller degree. This was most clearly demonstrated by the fact that the scroll centrifuges were ultimately determined to achieve optimum performance at feed rates of approximately half of what might normally be expected from machines of the size tested. In addition to the factors of cost, all machines were subject to the requirement that suspended solids recovery be at least 90 percent, so that the liquid sidestream could be handled without excessive degradation of the JWPCP effluent.

IV. RESULTS AND DISCUSSION

A. BELT FILTER PRESS PERFORMANCE

Belt filter presses can be operated to achieve a wide variety of results, by varying such parameters as belt speed, belt tension, polymer dosage, and auxiliary high pressure zone features. A series of experiments were performed in which these parameters were systematically varied to determine the effects on cake dryness and suspended solids recovery. These experiments clearly indicated that a number of tradeoffs can be made among the factors that affect system cost and performance.

1. Variation of Belt Speed and Polymer Dosage

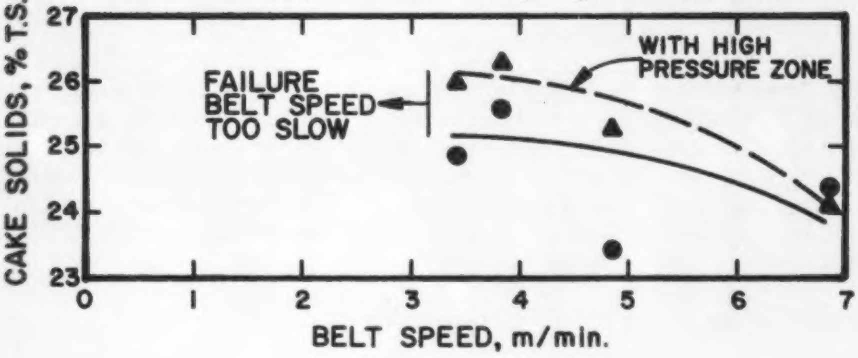
Belt speed directly affects cake solids and suspended solids recovery, and indirectly affects the required polymer dosage. A series of tests were conducted with screened digested primary sludge at a feed rate of 32 m^3/h (140 gpm) and different polymer dosages. Figure 5 shows that by lowering the belt speed, 1 - 3 percent drier cakes can be produced. Reducing the belt speed causes the sludge level to build up in the gravity drainage zone. Minimum belt speed is limited by the ability of the side baffle seals to contain the sludge within the gravity drainage zone; failure occurs when the sludge runs off the belts and is lost to the filtrate trays. Increasing the polymer dosage has a dual effect on cake solids. At any given belt speed, more water is released from the sludge because of the improved flocculation. In addition, belt speed can be lowered before failure occurs in the gravity drainage zone, further increasing cake dryness.

Figure 6 shows the effects of belt speed on solids recovery. In a belt press, solids are lost in two ways. They may either pass through the belt and appear in the filtrate, or they may adhere to the belt and be removed by the washwater. The results at moderate and high polymer dosages are typical for a sludge that is adequately flocculated. Increasing the belt speed exposes a given amount of sludge to more belt area. Since each square meter of belt traps a relatively uniform amount of solids after scraping, the high belt speed causes a greater amount of sludge to be lost in the washwater. Similarly, when using the high pressure zone features which apply external pressure on the belts, drier cake solids can be achieved, but additional solids will be squeezed into the belts and lost in the washwater. In contrast, at the minimum polymer dosage, significant amounts of unflocculated solids passed through the belt with the filtrate when belt speed was lowered, overshadowing the normal advantage achieved by lower belt speed.

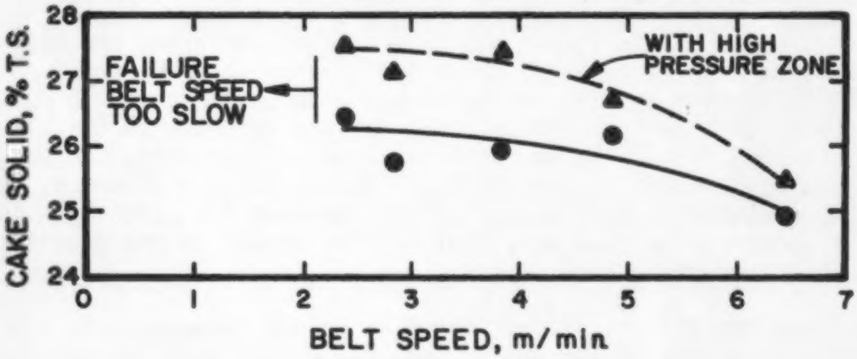
A further effect that was observed during the testing related to hydraulic capacity. Stable operation could be maintained at higher feed rates to the belt filter press by increasing the belt speed. However, other parameters can suffer from the increased belt speed, as described above.

In summary, lower belt speed can be used to maximize cake dryness and recovery in many cases. If it is desired to use the bare minimum of polymer or maximize hydraulic throughput, higher belt speeds may be required. Cake

LOW POLYMER DOSE 3.8g/kg (7.5 lbs/ton)



MODERATE POLYMER DOSE 4.9g/kg (9.7 lbs/ton)



HIGH POLYMER DOSE 7.5 g/kg (15 lbs/ton)

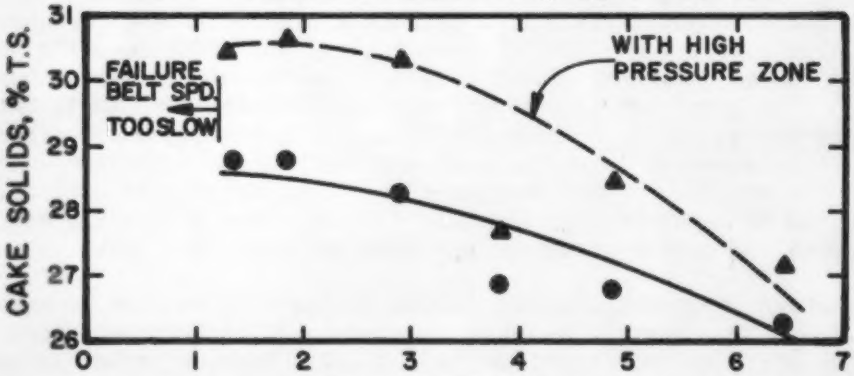
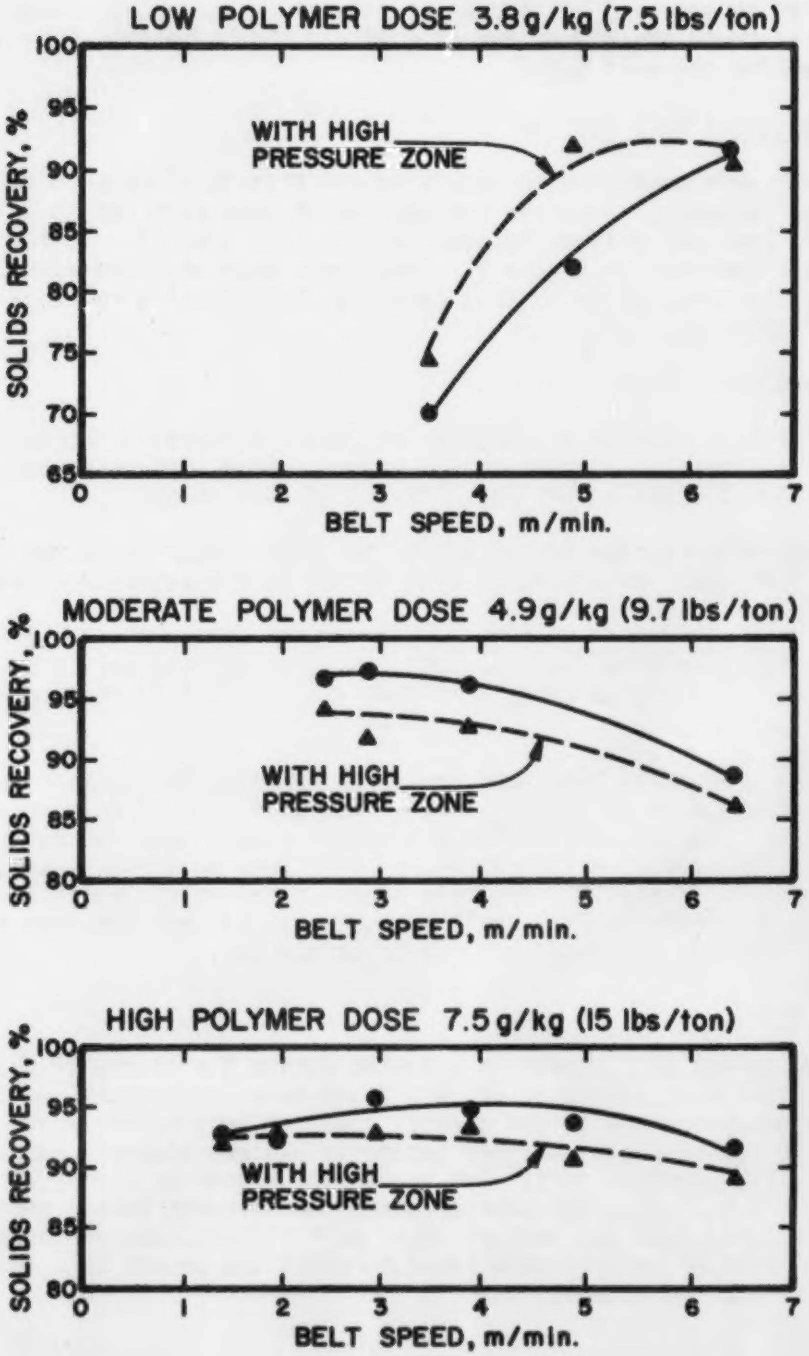


Figure 5. Effect of belt speed on cake solids using screened primary digested sludge.



LOW POLYMER DOSE 3.8 g/kg (7.5 lbs/ton)

MODERATE POLYMER DOSE 4.9 g/kg (9.7 lbs/ton)

HIGH POLYMER DOSE 7.5 g/kg (15 lbs/ton)

Figure 6. Effect of belt speed on solids recoveries using screened primary digested sludge.

dryness can be improved by increasing polymer dosage above the minimum required for stable operation, and this effect can be maximized by simultaneously lowering the belt speed.

2. Variation of Belt Tension

Experiments were conducted to determine the effects of belt tension on cake solids and recovery. Screened DPS was run at feed rates of 25 - 39 m^3/h (110 - 170 gpm) and polymer dosages of 3.5 - 5.0 g/kg (7 - 10 lb/ton). The results are depicted in Figure 7. Tests were made with and without the high pressure zone, which in this case uses two opposing rollers to cause a "wringer" effect.

Three trends are clear:

1. Higher belt tension produces drier cake. A fourfold increase in tension resulted in cake solids roughly three percent higher, equivalent to a ten percent reduction in dewatered cake volume.

2. Higher belt tension causes a loss in solids recovery on the order of 2 - 5 percent, primarily because solids were pressed into the belt and removed with the washwater.

3. High pressure zone features further increased cake solids by 1 - 2 percent, but caused additional solids to be lost in the range of 1 - 5 percent.

When blends of DPS and DWAS were dewatered, similar trends were noted, but as the percentage of DWAS in the blend increased, the effects became less significant. This is because the biological sludge does not have the structure to withstand much pressure, and as belt tension is increased, the sludge will eventually squeeze out from the edges of the belt. For sludge blends consisting primarily of DWAS, belt tension must be kept low, and high pressure zone features are not likely to be of any use.

3. Belt Weave

Several different belt weaves were tested during the course of the study. A very fine mesh belt tested on DPS was found to blind quickly, so that inadequate drainage occurred in the gravity zone. A very coarse weave belt necessitated the use of higher polymer dosage to achieve adequate recovery in comparison to the standard belt. Two belts were tested on blends of DWAS and DPS, and, unexpectedly, the coarser weave demonstrated better performance. It was concluded that any evaluation of belt filter press performance should include a trial of several belt types to match the proper belt to the individual sludge characteristics.

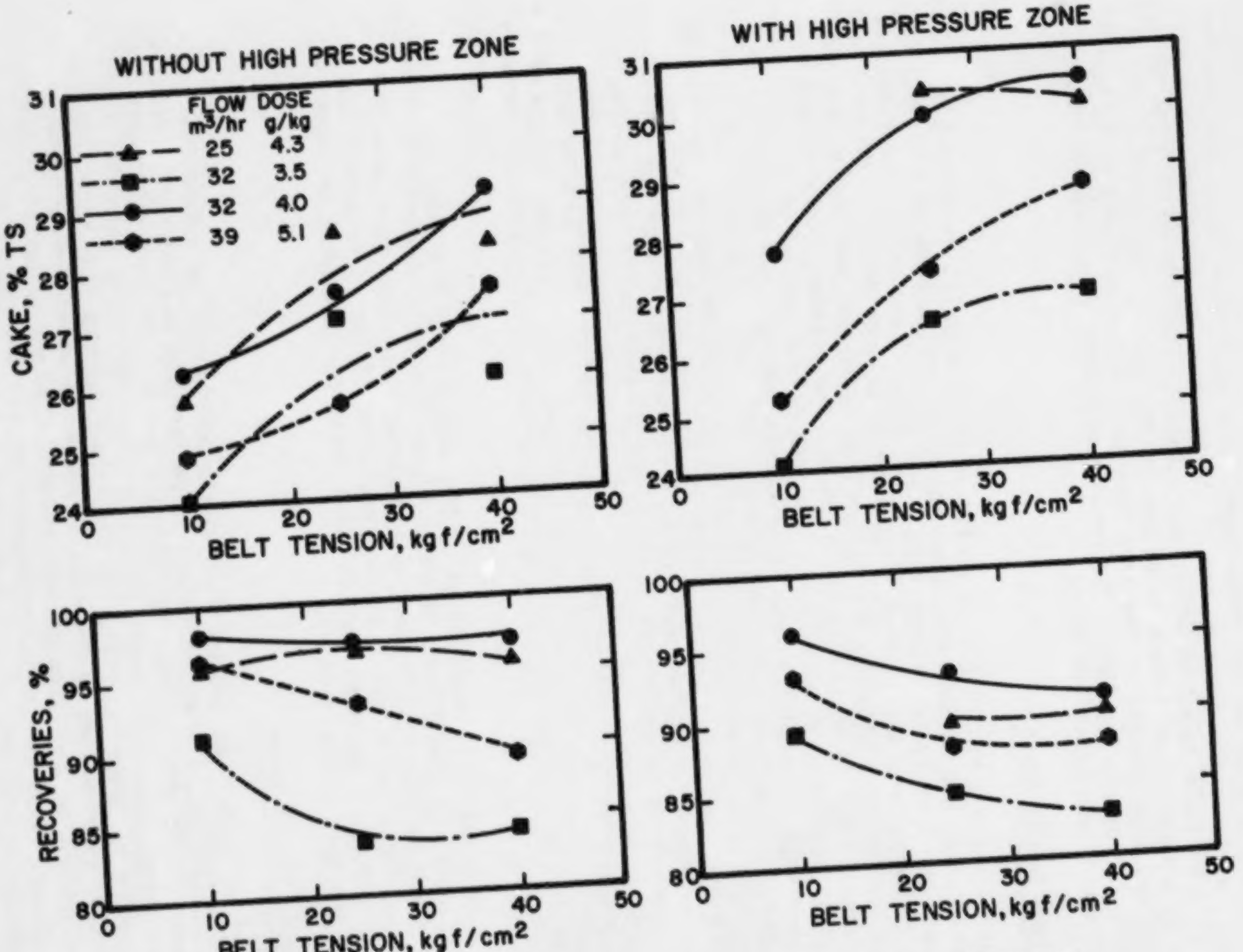


Figure 7. Effects of belt tension on Winkle Press performance using primary digested sludge

B. LOW SPEED SCROLL CENTRIFUGE PERFORMANCE

When testing started on the centrifuges, both were of a conventional design. However, early in the program, one of the machines was modified by the addition of a variable speed backdrive and automatic controls to vary the scroll speed as a function of torque developed on the scroll; the scroll of this machine was also changed at the same time to a design believed to be more suitable for dewatering DWAS. The results with both fixed and variable scroll speeds are described below.

1. Fixed Scroll Speed

One centrifuge was run through a series of tests with DPS in which bowl speed was varied from 900 to 1200 rpm, pool depth was varied from 19.2 to 19.7 cm, and feed rate from 23 to 68 m^3/h (100 - 300 gpm). Cake solids and suspended solids recovery were analyzed as a function of polymer dosage. The most successful results were obtained at maximum bowl speed and pool depth, and some of the results are shown in Figures 8 and 9. The recovery curve showed a clearly-defined break at about 2.5 - 3.0 g/kg (5 - 6 lb/ton) of polymer. Below this dosage, flocculation was insufficient and recovery dropped rapidly. Above this, recovery was consistently high. This held true until the flow rate was raised to 57 m^3/h (250 gpm), at which point the machine appeared to be overloaded, and nearly twice as much polymer was required to achieve acceptable recovery. At the lower flow rates, once the sludge was sufficiently flocculated, there was little to be gained by addition of more polymer. Cake solids fluctuated in the range of 21 - 23 percent regardless of feed flow or polymer dosage. The optimum results on DPS for this machine were thus found to be a feed rate of 45 m^3/h (200 gpm), and a polymer dosage of 3 g/kg (6 lb/ton), producing a cake with 22 percent total solids and recovery of 96 percent.

Similar data trends were demonstated by the second centrifuge that was tested. However, by comparison, it had a higher hydraulic capacity, a slightly lower polymer requirement, and produced cakes that were several percent lower in total solids. At the conclusion of the fixed scroll speed centrifuge tests, it appeared doubtful that these devices would be competitive with the belt filter presses.

2. Variable Scroll Speed

When the first centrifuge was modified by the addition of an automatically controlled backdrive, a dramatic change in performance was observed. Results of the tests on DPS are included in Figures 8 and 9. Polymer requirement to achieve excellent recovery changed only slightly. Hydraulic capacity of the machine dropped to about two-thirds of the former optimum flow rate. Most significantly, total solids of the cake increased by about four percent. This is equivalent to a fifteen percent reduction in the volume of dewatered sludge produced. This change in performance brought the centrifuge into direct competition with the results achievable with the belt filter press. The automatic backdrive also made a marked improvement in the stability of the centrifuge's performance, as will be discusseed in a later portion of this paper.

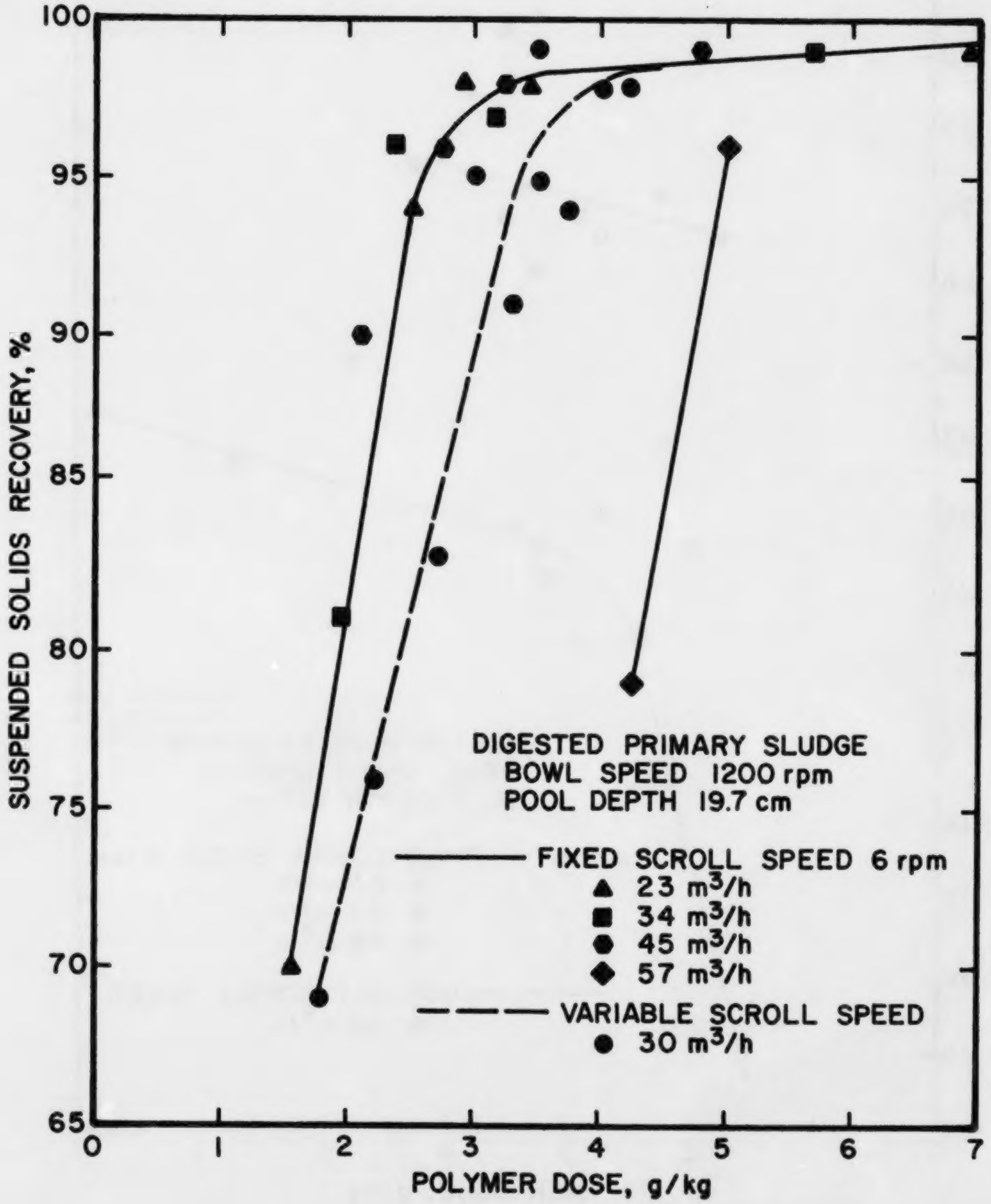


Figure 8. Low Speed Scroll Centrifuge - Recovery vs. Polymer Dosage.

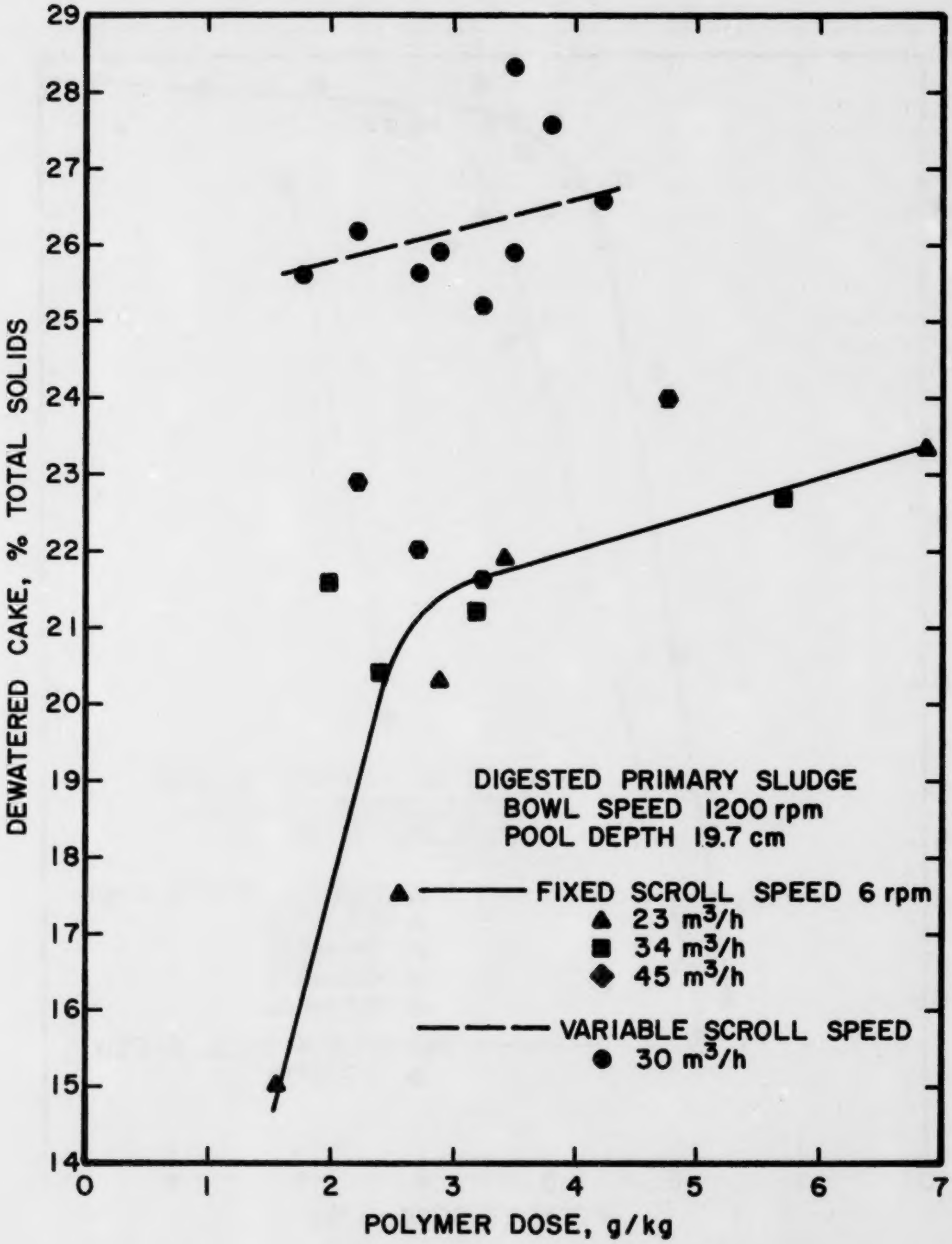


Figure 9. Low Speed Scroll Centrifuge - Cake Solids vs. Polymer Dosage.

Automatic control of the scroll differential in proportion to scroll torque also provides an additional means to optimize performance in accordance with the user's needs. When the automatic controller is set to maintain a higher scroll torque, the centrifuge builds up an increased inventory of solids within the pool volume. The increased retention of solids in the bowl results in a drier cake, but less pool volume is available for clarification. To compensate for this, additional polymer must be added to improve flocculation and maintain solids recovery. In an experiment with DPS designed to test this phenomenon, it was found that cake solids increased in a linear fashion from 26 percent to over 29 percent as scroll torque was increased; over the same torque range, polymer dosage had to be increased by approximately 1.2 g/kg (2.4 lb/ton) in order to maintain adequate recovery. This demonstrated that the setting of scroll torque could be used as a means to trade off cake dryness versus polymer requirement.

C. PERFORMANCE COMPARISON

Most of the testing to establish the operating characteristics of each machine was done using DPS as described earlier. A number of tests were also run using blends of DWAS and DPS containing from 25 percent to 70 percent DWAS on a dry solids basis. A summary of the optimum results from all of the tests are shown as a function of sludge composition in Figure 10. As shown earlier, a variety of results can be achieved with each device. The values selected for this figure were influenced by the cost analysis which established the relative importance of cake dryness, polymer dosage and flow capacity. Repeatability of the data was also considered, and greater weight was placed on data taken during simultaneous tests, in which the effects of variation in sludge quality were mitigated. All of the results shown for the centrifuge were achieved with the unit employing the automatically controlled backdrive. The optimum belt filter press results were also achieved with only one of the units tested.

What is most striking about the results shown in this figure is not the differences, but the similarities of performance. When total system costs to operate the two types of devices were compared, the differences did not appear to be significant when compared to the variability of the data used to establish costs. This ability of the centrifuge to match the performance of the belt filter press was not found to be the case until the introduction of the automatic backdrive.

The results for blended sludge containing more than 50 percent DWAS are rather questionable. All of the machines exhibited considerable difficulty in maintaining stable performance on sludges containing a high percentage of DWAS, requiring high polymer dosages and achieving recovery below the minimum acceptable (90 percent). On the belt filter presses, it was difficult to contain this sludge within the belts. This was somewhat unexpected, because pilot testing of the belt filter presses had shown acceptable performance on a blend containing 70 percent DWAS.

Smaller presses (0.5 to 1.0 m belt width) from both manufacturers supplying the large scale units had been tested during the pilot studies. Although the

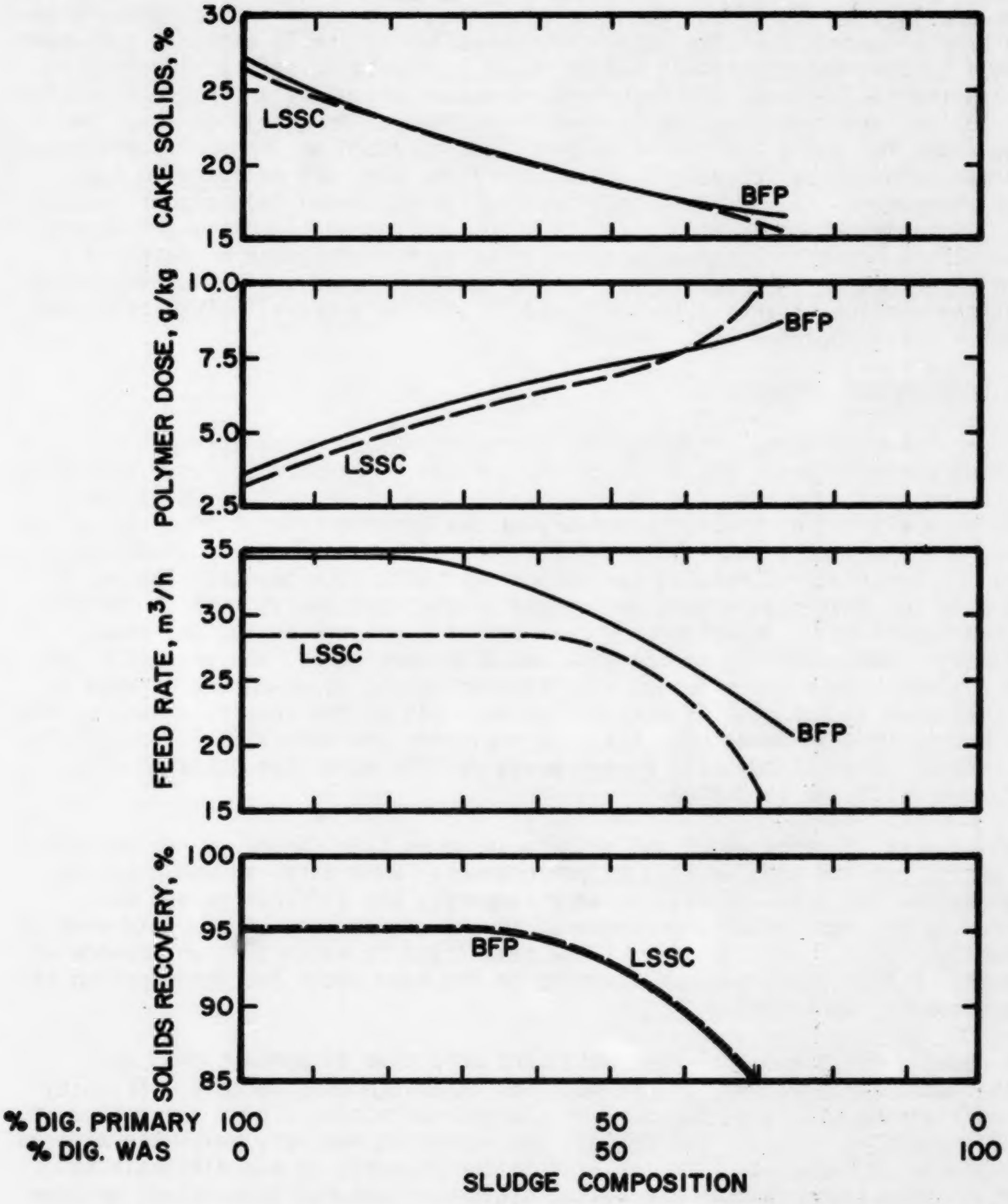


Figure 10. Summary of dewatering performance for Belt Filter Press vs. Low Speed Scroll Centrifuge.

term pilot study is used, these units were both normal production size units that were almost identical to the larger units with the exception of belt width. The larger units had difficulty matching the performance of the smaller units for all types of sludge tested, especially in terms of cake dryness. A possible explanation is that the smaller units could more effectively tension the belt, and contain the sludge within the belts in a manner that was difficult to reproduce in the larger units. There is some inherent limitation in the belt width that may be used. Wider belts should be woven of larger fibers to withstand distortion and maintain uniform high tension; however, the fiber size is also governed by the required filtration properties of the belt. Belt filter press manufacturers for the most part are using the same type of belt fabric on all size machines, and this may result in relatively poorer performance on the larger units.

Although a number of different scroll centrifuges were also tested in the pilot studies, these tests could not be matched up with the large scale studies to provide information on the effects of scale-up.

There are some other differences in performance that are not evident from the results shown thus far. The single highest value for cake solids achieved by the belt press on DPS was 33 percent, whereas with the centrifuge, the highest value was just under 30 percent. These values are representative of only a few test runs, and are not likely to be achievable on a long term basis. The very dry cakes were also produced at the expense of high polymer dosage, particularly in the case of the belt filter press. However, there does appear to be some evidence that, taken to the extremes, the belt press can produce a slightly drier cake.

The reaction of the machines at minimum polymer dosage is also quite different. Under normal operation, there is a certain amount of variation in the sludge flow rate, and even more so in the suspended solids concentration of the sludge, that cannot be avoided. When a minimum polymer dosage is chosen for the average conditions, there are likely to be periods when the sludge is under-conditioned. When under-conditioning occurs on the belt filter press, the sludge will drain poorly in the gravity zone, and the sludge depth will increase until the seals cannot prevent it from flowing off of the edges of the belt. Similarly, a poorly conditioned sludge can squeeze out from between the belts in the pressure zones. These conditions result in a drastic loss in solids recovery, and some decline in the dryness of the cake. Under similar conditions, the centrifuge does not fall into a failure mode, but suffers a more gradual loss in recovery as the solids level builds up in the pool. The machine equipped with the automatic backdrive was also found to automatically compensate for such variations, and maintain the dryness of the cake. It is for these reasons that Figure 10 shows that the belt filter press requires a polymer dosage about 0.5 g/kg (1 lb/ton) higher. The additional polymer is necessary to achieve stable performance on the belt filter press because intermittent under-conditioning causes a much more severe loss of recovery (from sludge spillage) than occurs with the centrifuge.

Although both types of devices were able to achieve recovery well above that required for the JWPCP, there was some difference that might be significant in other situations. It was found that the centrifuge could be adjusted to achieve in excess of 98 percent recovery on digested primary sludge with a moderate increase in polymer dosage. Under similar conditions, the belt filter press appeared to be limited to a maximum of about 96 percent recovery, which by comparison would leave twice the amount of solids remaining in the liquid sidestream. The practical limitation on the belt filter press is that, after solids are scraped from the belt, there will always be a small amount pressed into the belt weave which will be removed with the washwater. The centrifuge does not have a parallel limiting factor.

The factor of flow capacity is frequently the most controversial, because of the role it plays when equipment is acquired by a competitive bid process. Note that the belt filter press represented in Figure 10 was shown to have an optimum capacity of 34 m^3/s (150 gpm) on DPS, while the centrifuge capacity was selected to be only 28 m^3/s (125 gpm). Of these two devices, the centrifuge has a first cost roughly 50 percent higher. However, it is notable that this centrifuge (before addition of the automatic backdrive) was operated successfully at flows as high as 68 m^3/s (300 gpm). It might also be considered that the second belt filter press tested was found to have an optimum capacity of only 22 m^3/s (100 gpm), although both presses were of similar belt width. The wide variation in capacity for seemingly comparable machines points out that the capacity for a particular machine can only be established by field testing, with all of the other operating conditions clearly specified. It must be recognized that the cost to manufacturers will be determined only by the number of machines he must supply, but that this may be only a small part of the total cost to the user for the life of the project. It is recommended that bidding situations should be avoided which give a large advantage to the manufacturer who can supply the least number of machines.

V. OPERATION AND MAINTENANCE EXPERIENCE

A. TOTAL OPERATING TIME

The various factors that were found to affect operation and maintenance are the subject of this portion of the paper. During the test program, one of the belt filter presses was operated for a total of 1600 hours. The second belt filter press was operated only 350 hours, because of belt failures which will be described below.

The centrifuge equipped with the automatic backdrive was acquired on a long term lease after normal testing concluded, and was then placed into regular service on digested primary sludge. By February, 1980, more than 3500 hours had been logged on the machine. The second centrifuge was operated for a substantially shorter time because it was not placed into regular service.

B. EASE OF OPERATION

1. Belt Filter Press

The belt filter press provides an advantage to the operator because all parts of the process can be readily seen and understood, and the results of operational changes are quickly apparent. It was found that the belt filter press could be started and placed into stable operation within a few minutes. A minimally acceptable operation can be maintained by an operator of relatively low skills. However, the efficiency of operation can be greatly affected by the choice of belt speed and belt tension, use of high pressure features, control of polymer injection and mixing, adjustment of washwater, and a number of other variables which are peculiar to each machine. The use of these features to their maximum advantage can be easily ignored if the operator is not interested in achieving the most economical operation.

The sensitivity of the belt filter press to changes in sludge conditioning was touched upon earlier. When the belt filter press is operated near the minimum polymer dosage, periods of under-conditioning are likely to occur if there are variations in the flow, suspended solids concentration, or sludge quality (such as the percentage of WAS in combined sludges). Under-conditioning usually results in a drastic loss of solids recovery, because sludge spillage occurs from the sides of the machine. Although many manufacturers provide some kind of sensing device to maintain certain performance indicators, they are of questionable efficacy because sludge loss can occur almost anywhere in the machine. It is recommended that users of belt presses provide a large volume of holding capacity (e.g., one day storage) to dampen fluctuations in sludge quality and allow feed flow to be carefully controlled. Continuous monitoring of flow and feed suspended solids concentration with automatic compensation for polymer dosage may also be effective. If variations in the sludge feed cannot be closely controlled, intentional overdosing with conditioning chemicals may be the only answer.

2. Centrifuge

Operation of the centrifuge by the unskilled operator is made slightly more difficult because he cannot see what is taking place inside the machine, and must use a certain amount of deductive logic to understand what happens when he makes a process change. Also, a condition such as poor centrate quality is more easily overlooked, since the process is fully contained; by comparison, the signs of poor performance on the belt filter press are readily visible.

On most centrifuges, changes in the bowl speed, scroll differential or pool depth require some disassembly of the machine, and so are less likely to be attempted than the type of adjustments that are applicable to the belt press. As a result, the centrifuge requires a more extensive program of pretesting to select the correct machine settings before it is placed into normal service.

The use of the automatically controlled backdrive greatly simplifies the operation of the centrifuge, particularly the maintenance of stable performance. The programming of the automatic controller makes startup very easy, by allowing the centrifuge to build up the correct inventory of solids and then adjusting the scroll speed to discharge them at the appropriate rate. The use of scroll torque to control scroll speed is very successful in compensating for variations in feed flow and suspended solids concentration.

During the testing program, feed flow jumped to as high as twice the normal flow rate for short periods. When this happened, the centrifuge suffered a temporary loss in recovery, but maintained excellent cake dryness. The automatic backdrive provides a means for maintaining stable operation which does not yet appear to be available for the belt filter press. Since the process is fully enclosed, the problem of sludge spillage is not a factor, as it is with the belt filter press. Where variation in sludge quality can cause considerable variation in polymer requirement, such as in the dewatering of combined primary and waste activated sludges, it is still recommended that storage be provided ahead of dewatering to equalize the fluctuations in sludge quality.

C. MAINTENANCE OF WEARING PARTS

1. Belt Filter Press

The belt filter press has a large number of moving parts, including bearings, rollers, and hydraulic pistons for belt tracking. Nevertheless, most parts are small enough and easily accessible, so that even most small facilities would have no difficulty in maintaining these parts. Specification of long life parts with effective seals to prevent water intrusion would be expected to keep replacement costs to an acceptable level.

The major concern with this type of device is the matter of belt life. Although it was found that a set of belts could be easily replaced in a matter of hours, the cost for a set of belts for a two meter machine is likely to be $3,000 - $5,000, and the Districts' experience indicates that the life is likely to be only about three to six months in continuous service.

During the course of the testing of the press that was operated 1,600 hours, three belt sections failed, amounting to one full set of belts. One belt was torn immediately after startup, apparently due to an improperly adjusted scraper blade. The other two occurred near the end of the testing. The belt manufacturer stated that these failures were due to a weaving problem that caused reduced strength and that had since been corrected; however, by this time, a number of frayed areas had appeared, indicating that the material was reaching the end of its life under any circumstances.

The second press, which was operated only 350 hours, also damaged three sets of belts. This machine exhibited difficulty in obtaining a uniform distribution of sludge on the belts at the end of the gravity drainage zone. This poor distribution apparently allowed the belts to wrinkle as they passed through the rollers, and the fabric was weakened at these points and eventually torn. The poor distribution of sludge was believed to be caused by improper cleaning of the lower belt by the washwater system. Although a number of different belt types and spray systems were tried, this problem was never solved.

Scraper blades on the first machine were found to be worn out after 1,400 hours. Sharp, well adjusted scraper blades are critical to the maintenance of acceptable recovery. The blades must be adjusted to ride closely on the belt, but if too tight, can catch any irregularity in the belt and tear it.

The cost and time to replace scraper blades is considered to be minor. However, the condition and adjustment of the blades is a critical feature that should be checked frequently.

2. Centrifuge

The moving parts of the centrifuge are relatively fewer and larger. Handling of the large, rotating assemblies would be of most concern at small plants. In a large plant it is practical to provide suitable overhead or mobile cranes to handle these assemblies, and it is also likely that sufficiently skilled workers would be available for on-site maintenance. At small plants, some factory maintenance of the rotating assemblies may be required.

The Districts have operated high-speed, countercurrent flow scroll centrifuges for over a decade, providing a greater base of experience to evaluate this type of device. In that time, it has been found that wear of the scroll tips is the only item of major concern. Bearing failures have been almost nonexistent. Scroll tips have been hard-faced with new weld material approximately every 2,000 hours when used in continuous service on DPS. All repair has been performed on-site. A test of a scroll with ceramic tipped flights is also now in progress. To date, this has exhibited a life of at least eight times that for conventional hard-facing. Since the new centrifuges operate with a scroll differential an order of magnitude lower than the old centrifuges, a further increase in life is expected. Overall, scroll maintenance is expected to be an easily manageable problem, particularly for a facility the size of the JWPCP.

The automatic backdrive equipment includes high pressure hydraulic systems, and conventional electronic controllers for closed-loop control. Neither have exhibited any particular maintenance problems, although they do create the requirement for some degree of skilled maintenance.

D. NOISE AND VIBRATION

Noise and vibration are of little concern for a belt filter press installation. Noise from a scroll centrifuge is quite noticeable, but not so high as to require ear protection for workers. Vibration is also noticeable. This caused a small problem, because the automatic controller was mounted on the same platform as the centrifuge, causing some electrical connections to loosen up. Since this was a temporary installation, no provision had been made to shield the controller from vibration. In a permanent installation, it is recommended that sensitive electronic controls be isolated from vibration. Structural components and fasteners should be designed to resist vibration. The rotating assemblies of the centrifuge may require a periodic rebalancing, especially after rebuilding of scroll tips, to avoid excessive bearing wear.

E. REQUIREMENT FOR PRE-SCREENING

Belt filter presses were tested with both screened and unscreened PDS. Screening was provided by a rotating cylindrical strainer with 0.25 cm (0.1") slots. When operated on unscreened sludge, fibrous materials were found to quickly build up on scraper blades and in the washwater unit enclosure. Larger objects were also found in the sludge which appeared capable of puncturing the belts. Prescreening of the sludge is thus considered an important adjunct for a belt filter press facility. Operation of the scroll centrifuge on unscreened sludge has not presented any problems.

F. CLEANUP

Operation of a belt filter press can be rather messy, particularly when failure due to under-conditioning causes sludge to run out from the sides of the machine. One of the presses also had a constant problem of filtrate and washwater spilling from the sides of the machine, even during stable operation, and a fine mist escaping from the spray wash unit. Scraper units and filtrate trays are sites where solids build up. It is recommended that a belt press installation be designed with daily washdown by hosing in mind. Good drainage and safe walking areas are thus important.

Since the centrifuge is a fully enclosed machine, cleanliness is much less of a problem. Problems are confined to conveyor areas, which should be designed for frequent washdown for both centrifuges and belt filter presses.

G. ODOR EMISSION

The belt filter press generally has large free surface areas where the sludge is exposed to the atmosphere, particularly in the gravity drainage zone. A considerable amount of odors can be emitted, especially if the sludge is inadequately digested. Also, a large amount of steam can be released if the sludge is still hot from anaerobic digestion. If the presses are located within a building, large vent hoods over each press would be recommended, and the ventilation system should be designed for good drainage of condensation.

The centrifuge is an enclosed machine, so odor emission is less of a problem. A vent is necessary because of the windage from the rotating assembly, and the duct work from this vent should also be designed for drainage of condensate.

H. AUXILIARY WATER REQUIREMENTS

The belt filter presses require an auxiliary supply of water for washing the belts. The flow rate required was found to be roughly 50 to 100 percent of the flow rate of sludge to the machine, and the pressure is typically 7 kgf/cm (100 psi) or more. Since the cost of the water is relatively minor, the auxiliary water is only a problem if some kind of treatment must be provided for the liquid sidestream.

The centrifuge requires only a small amount of auxiliary water for cooling.

I. ENERGY REQUIREMENTS

The major power requirement for the belt press derives from the pumping of washwater. Overall, the low speed scroll centrifuge requires about three times as much power when operated at the capacities indicated in this paper. In spite of this higher demand, the power costs were minor when compared to the other costs of dewatering and disposal.

VI. CONCLUSIONS

1. The belt filter press and the low speed scroll centrifuge with the automatically controlled backdrive can achieve equivalent performance on digested primary sludge, and blended sludge containing as much as 50 percent by weight of digested waste activated sludge. For both machines, dewatering of combined sludge containing a high percentage of DWAS resulted in low recovery, high polymer requirement, and a dewatered cake that was difficult to handle.

2. The addition of the automatically controlled, variable speed backdrive to the centrifuge resulted in a great improvement in cake dryness at the expense of hydraulic capacity. This change placed the centrifuge in direct competition with the belt filter press.

3. Both the belt filter press and the low speed scroll centrifuge can achieve a wide variety of results on any given sludge, depending on the selection of feed rate and polymer dosage, and the adjustments of various mechanical features. This was found to be especially true on the belt press, which allows for easy adjustment of several mechanical variables. The automatically controlled backdrive provides a similar flexibility for the centrifuge.

4. Both types of device were relatively easy to start up and operate. Belt filter press operation is easier to understand for the novice because all parts of the process are clearly visible. The belt filter press was more subject to instability and drastic failure characterized by sludge spillage when under-conditioning of sludge occurred. The automatic backdrive contributed significantly to the stability of performance achieved by the centrifuge.

5. The major concern for maintenance of the belt filter press is belt life. Belts would be expected to last three to six months in continuous service, and cost $3,000 to $5,000 per set for a two meter machine. For the centrifuge, scroll tip replacement is the major maintenance item. Ceramic hardfacing would be expected to last several years in continuous service. Replacement costs are comparable to the costs for one set of belts on the belt filter press. If the replacement could not be performed onsite, additional costs would be incurred for transportation to the factory. The centrifuge's large, rotating assemblies may be more difficult to maintain at a smaller facility than the type of components that make up a belt filter press.

6. Belt filter press operation can be relatively messy, and emit significant amounts of odor and moisture. Scroll centrifuge operation is subject to moderate noise and vibration.

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THERMAL CONVERSION OF SLUDGE IN A MULTIPLE HEARTH FURNACE, USING A SUB-STIOCHIOMETRIC SUPPLY OF OXYGEN

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INTRODUCTION

In June 1975, the Interstate Sanitation Commission (ISC) released a report (1) by an engineering firm, Camp, Dresser, and McKee (CDM), which evaluated the alternatives for sludge disposal for the New York Metropolitan Area if ocean disposal were no longer permitted. The engineering firm concluded that the most cost-effective alternative was incineration in multiple hearth furnaces with disposal of sludge ash to landfill. In the New York Metropolitan Area, the suggestion that all sludge be incinerated would be unpalatable. Public experience with municipal solid waste incinerators and with incinerators from multi-dwelling housing units has been bad in this area, and an adverse reaction from the public sectors would likely develop even though sludge incineration would be much more likely to be free of nuisance than solid waste incineration. At least in part to avoid this potential obstacle (in this writer's opinion), the authors of the report suggested that "pyrolysis" could be used to process the sludge. The "pyrolysis" could be conducted in a multiple hearth furnace. The technology was not yet developed and could conceivably fail. In that case, the multiple hearth furnaces could be used to incinerate the sludge. In a second more detailed report (2), the authors proposed a "pyrolysis" process that followed closely the method suggested by Majima, et al. (3), in which sludge was predried before it entered a multiple hearth furnace for the pyrolysis step.

Shortly after the CDM report (1) to the ISC became available, this writer was reviewing plans for sludge disposal at the Blue Plains wastewater treatment plant in Washington, D.C. One of the disposal options seriously being considered at this time was incineration. Local codes required that the incinerator off-gases be passed through an afterburner and the design engineers proposed an exit temperature of 870 °C. Combustion of a high grade fuel--oil or gas--would be required in the afterburner. Fuel costs would be excessive and a valuable national resource would be consumed. The writer realized that the "pyrolysis" mode of operation of a multiple hearth furnace (MHF) would produce a burnable gas at the top of the furnace. Afterburning very likely could be achieved without the need for high quality fuel. As a consequence, a project to investigate "pyrolysis" in a MHF was included in the Municipal Environmental Research Laboratory (MERL) research plan.

The MERL Laboratory contacted the Interstate Sanitation Commission (ISC) and found them eager to pursue an evaluation. The ISC interests were slightly different from MERL's. They were interested in the potential of "pyrolysis" for reducing the emissions of toxic materials to the environment. The investigation was planned to meet both objectives. The facilities to be used were the pilot plant of Nichols Engineering Company at Belle Meade, N.J. Sludges from the New York Metropolitan Area were to be tested. Experimental work was carried out in two phases, one completed in 1977, and the second in 1979. Final reports on the entire study were received in late 1979.

The processes for "pyrolyzing" sludge envisioned and actually carried out do not fit the conventional sense of the term pyrolysis. This has caused a degree of controversy in the United States over calling the proposed processes "pyrolysis." Ordinarily, pyrolysis means decomposition in the absence of oxygen. There are numerous pyrolytic processes: thermal cracking of petroleum, destructive distillation of wood, pyrolysis of coal, pyrolysis of solid waste (using indirect heat transfer). The process configurations proposed for sludge introduce air or oxygen into the reaction vessel. The gaseous products of pyrolytic decomposition of sludge are partially combusted within the reactor to supply heat, and in some operating modes the sludge solids are eventually exposed to an oxygen-rich atmosphere. Uncombusted gaseous products are generally completely combusted in an afterburner. Overall results superficially resemble incineration. In fairness, it should be said that the specific process conditions may produce a solid residue as well as particulate and gaseous pollutant streams much different from those produced by incineration.

Objectors to the term "pyrolysis" evidently suspect that a new name has been invented for an old process--that "pyrolysis" is essentially incineration with the same old pollutional potential but with a new name that carries with it the implication that there is no release of gaseous pollutants. On the other hand, alternative names such as starved-air combustion or partial combustion do not convey the idea that the processes offer better control of the reaction at the solid-gas interface than conventional combustion. As a compromise this presentation refers to the "pyrolysis" processes for sludge as thermal conversion with sub-stoichiometric oxygen supply (TC-SSOS).

Publications by Japanese authors (3) indicated that sludge could be decomposed in a multiple hearth furnace (MHF) under conditions that did not expose the sludge to strongly oxidizing conditions, and that produced a combustible fuel gas at the top of the furnace. Because the sludge is not thoroughly combusted, it does not release all of its heating value; consequently, some strategem must be used to allow autothermal operation. These authors predried the sludge to about 70 percent solids. In our approach, it was decided to achieve the same goal by dewatering the sludge to a high degree by means of filter presses or diaphragm filter presses. Some supplemental fuel might be required in the MHF, but the large amount of fuel required for the afterburner would be eliminated.

The primary objectives of the experiments carried out by Nichols Engineering for the ISC were as follows:

1. Demonstrate that by controlling the air supply, a MHF can be operated to discharge a combustible gas at the top hearth and an inert residue from the bottom hearth. The combustible gas should have sufficient heat of combustion to burn completely in an afterburner, reducing unburned hydrocarbon to acceptable levels. (This operating procedure is referred to as thermal conversion, substoichiometric oxygen supply: TC-SSOS).

2. Investigate the air pollutional discharges and solid residues produced by TC-SSOS and compare them to operation of the MHF in the incineration mode.

As the following presentation indicates, the objectives were incompletely realized. Additionally, review and assessment of the air pollution data have not yet been completed. Nevertheless, much knowledge has been gained. New ways to process sludge in a MHF have been discovered that may possibly lead to useful solid products from sludge.

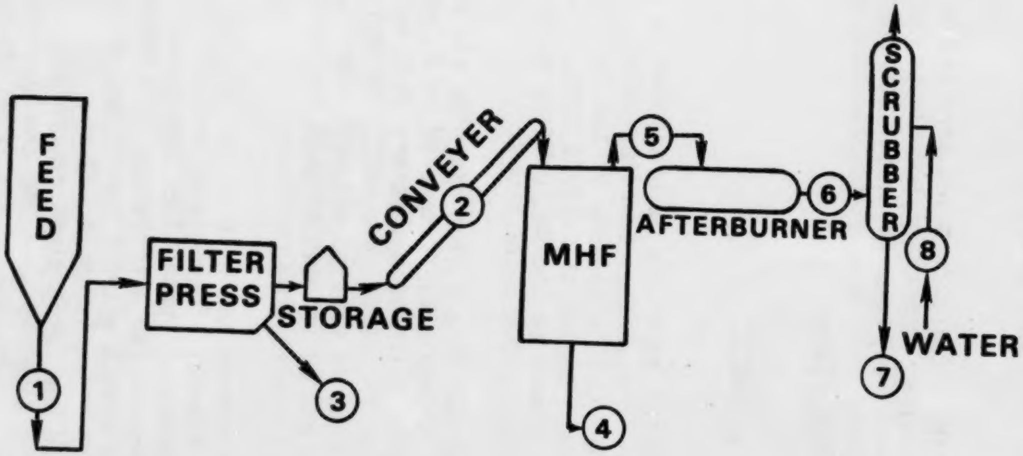
EXPERIMENTAL APPARATUS

A schematic diagram of the experimental apparatus is presented in Figure 1. Also shown on this diagram are the various sampling points. The preparation of the filter cake was carried out in advance of the furnace runs. During a four-day period preceding a series of furnace runs, sludge was delivered from the sewage treatment plants to the Belle Meade pilot plant by tank truck, dewatered, and stored in 200 liter drums.

MHF - the TC-SSOS runs were carried out in a 0.91 m diameter six hearth MHF, which is illustrated in Figure 2. Sludge cake is metered by a feed conveyor and enters the top hearth (Hearth 1) through a rotary star valve. The sludge cake is rabbled inward towards the center drophole, through which it falls to Hearth 2. Here the sludge cake is rabbled outward, where it falls through the drophole to Hearth 3, and so on down through the furnace. Final product exits from Hearth 6 into a sealed drum.

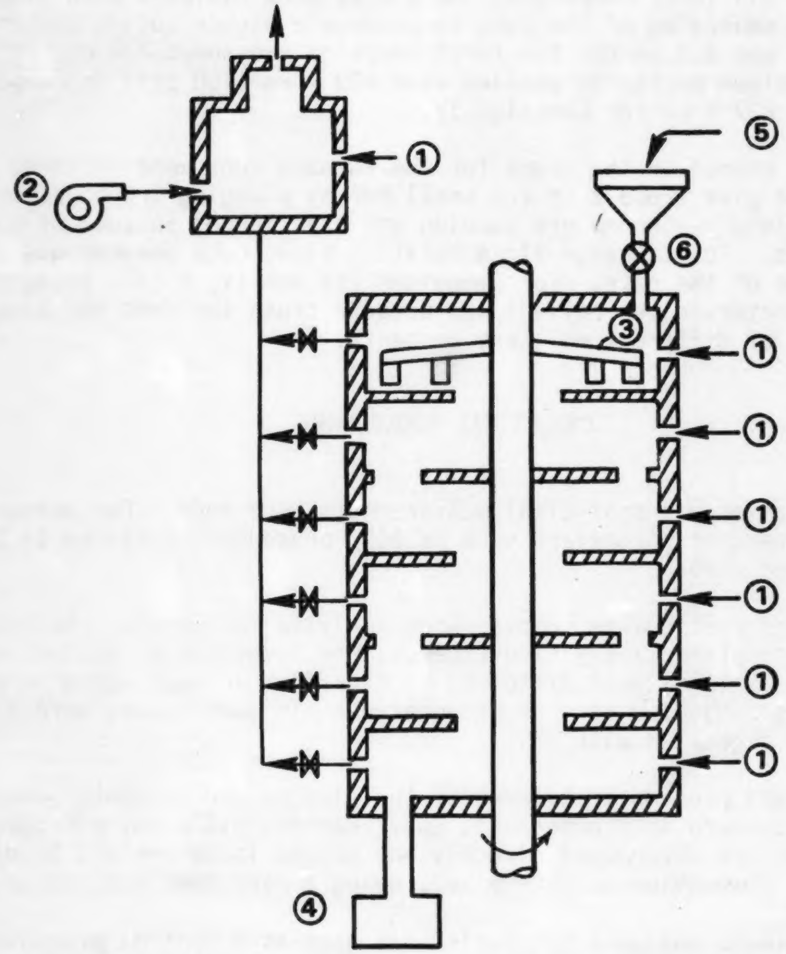
Each hearth is equipped with a gas burner and metered gas and air supply. Steam can be injected on each hearth. There is an exhaust line containing a damper (valve) from each hearth. These lines are joined in a common manifold leading to the afterburner. The afterburner is a refractory-lined chamber, 2 m^3 in volume, equipped with a gas-air burner and an auxiliary air supply. Actual hearth area in the furnace is about 3.1 m^2. Gas temperature can be measured on all hearths, and bed temperature can be measured on Hearths 3, 4, and 5. The temperature is also measured, in the approximate center of the afterburner. Gases from the afterburner are drawn through a scrubber where they are cleaned and cooled, and discharged to the atmosphere.

Filter Press - Preliminary determinations of dewatering rates were conducted with a small filter press with 0.3 x 0.3 meter plates. Sludge cake for the furnace runs was produced on a 1.2 x 1.2 m filter press. For


FEED
FILTER PRESS
STORAGE
CONVEYER
MHF
AFTERBURNER
SCRUBBER
WATER
1
2
3
4
5
6
7
8

1 Sludge sampling
2 Feed sampling
3 Filtrate sampling
4 Hourly furnace residue sampling
5 Analyses for SO_x, NO_x, and hydrocarbons
6 Particulate sampling
7 Scrubber water sampling
8 Influent water sample

FIGURE 1 ILLUSTRATION OF MATERIAL FLOW WITH SAMPLING POINTS INDICATED



1 Gas-air inlets, flows are metered

2 Low pressure air blower

3 Rotating rabble arm, 2 or 4 per hearth (only rabble arm on first hearth is shown)

4 Ash hopper

5 Charge fed by belt feeder

6 Rotary valve

FIGURE 2 FLOW SHEET OF BELLE MEADE MHF

the first of the four experimental campaigns, the plates were of conventional design; for all later campaigns, the plates were equipped with diaphragms that permitted squeezing of the cake to produce a higher solids content. Cake thickness was 3.2 cm for the first campaign and about 2.7 cm for later campaigns. Maximum pressures applied were 689 k Pa (100 psi) in Campaigns II and III and 827 k Pa for Campaign IV.

The cakes stored in the drums for the furnace runs were in large fragments and could give trouble in the small MHF by plugging dropholes or by burning incompletely--incomplete burning was experienced in some of the Campaign I runs. For Campaign II, a rotating blade cake breaker was used to reduce the size of the cake. For Campaigns III and IV, a cake breaker comprised of two motor-driven rollers was used to crush the cake and blend sludge batches of different moisture contents.

ANALYTICAL PROCEDURES

A great variety of analytical measurements were made. For measurements of typical wastewater parameters such as BOD, procedures outlined in Standard Methods (4) were used.

Sludges and particulate samples were analyzed for metals. Methods used were atomic absorption, X-ray fluorescence, and inductively coupled argon plasma (ICAP). Samples were dissolved by digestion in aqua regia in most cases. Samples subjected to high temperatures, in some cases, were also put in solution by digestion with HF.

Ultimate and proximate analyses of the sludges and residues were conducted using standard ASTM procedures (5). Carbon, hydrogen, nitrogen, sulfur, and ash are determined directly and oxygen is determined by difference. Heat of combustion is determined, using a Parr bomb and sodium peroxide.

The "proximate analyses for fuels" was used as a control procedure and was run hourly on sludge feed and residue. Proximate analyses determine volatiles (VOL), fixed carbon (FC), and ash. To determine volatiles, the solids are dried at 105 °C and then heated in the absence of air for 7-1/2 minutes at 949 °C. The percent volatiles in the loss in weight relative to the weight of the dried sample. To determine fixed carbon, the devolatilized sample is burned to ash at 949 °C. The net loss in weight is the fixed carbon. This determination also yields the percent ash. It is important to note that the volatiles determined by proximate analyses does not correspond to the volatile solids determination carried out by sanitary engineers. The sum of VOL and FC is essentially the same as the sanitary engineer's volatile solids. Reference made to volatiles (VOL) in this presentation refers to the volatiles of the proximate analysis for fuels and never to the sanitary engineer's volatile solids determination.

Particulate discharges were measured using standard EPA methods. All measurements were made on unabated discharges, that is, upstream from the particle removal devices. Total particulate catches were measured by the

EPA-5 method (6) and the SASS train (7). The SASS train not only gives total particulate but fractionates the particulates into size ranges. Experiments were also made with the Anderson sampler, which also fractionates the particulates.

Detailed consideration of these devices is not warranted here because discussion of air pollution results is not considered in this presentation. It is necessary to note that for both the EPA-5 and the SASS train, the sampled gases are withdrawn isokinetically, and are brought to an oven maintained at a temperature above the dewpoint of the gas (121 °C). The final filter, which is a glass fiber filter, reportedly removes particles down to 0.1 micrometer in size.

Analyses were also conducted for NO_x and SO_2. No further discussion is warranted here because results have not been reported in this presentation.

EXPERIMENTAL RESULTS

Filter Press Experiments

Preliminary experiments with the 0.3 x 0.3 m filter press were conducted to determine whether the sludges proposed for TC-SSOS could be dewatered at reasonable rates to sufficiently high solids contents. Results of unoptimized tests carried out at 689 k Pa (100 psi) differential pressure, using polymers and lime and ferric chloride as conditioning agents, are presented in Table 1. Solids contents above 26 percent (conditioning-agent free basis) were obtained in all cases. Of eight sludges, all but one could be dewatered either with polymers or with ferric chloride and lime. Better results were expected with fresher sludge, optimized dose of conditioning agent, and the use of a diaphragm filter press. Consequently, the thermal conversion tests were scheduled to proceed.

Prior to a series of thermal conversion tests, a large batch of dewatered sludge cake must be produced. Liquid sludge, delivered by tank truck is fed together with polymer solution or aqueous ferric chloride into a pug mill, where the two streams are mixed. The mixture is pumped to a holding tank which is kept well-mixed by an air lance. For ferric chloride-lime conditioning, the lime ($Ca(OH)_2$) is added to the holding tank. The sludge is delivered to the filter press by a diaphragm pump. Pumping is stopped when pressure reaches 480-550 k Pa (70-80 psi). Compressed air is applied to the diaphragms (689 k Pa) and the cake is squeezed for 5-15 minutes. Sludge cake is stored in drums 1 to 4 days before use.

Results obtained during the first three experimental campaigns are presented below:

TABLE 1

Preliminary Dewatering Experiments with Sludges from the New York Metropolitan Area[1]

Sludge Source	Type	Conditioning Agent Dose (WT %)		Percent Solids in Cake	
		Polymer	Lime and Ferric Chloride	Actual	Conditioning Agent-Free Basis
Coney Island, NYC	Digested	.72		39.9	39.9
			35	44.9*	37.6
26th Ward, NYC	Raw	0.20		26.8	26.8
			34.5	26.7*	21.3
26th Ward, NYC	Digested	0.65		25.1	25.1
			32	40.8	34.3
Cedar Creek	Raw	0.35		22.5	22.5
			20	31.9	28.1

1 - If amount of inert additive (such as a conditioning agent) is not too large, its addition will not affect thermal performance of a sludge in a MHF provided solids content on an additive-free basis is unchanged. Chemically conditioned sludges indicated by the asterisk will have poorer thermal performance than the corresponding polymer-conditioned sludge because additive-free solids content is lower.

SLUDGE	CAMPAIGN	% SOLIDS
Coney Island Digested	I	31.9
Coney Island Raw	II A	41.4
	II B	42.2
Rahway Raw	III	31.5
Jersey City Primary	IV	51.0

These results were obtained with polymer conditioning. In Campaign II B, a considerable amount of sludge was conditioned with lime and ferric chloride. Poor results were being obtained with polymer, so lime and ferric chloride were used instead. The sludge could then be filtered but solids content was not improved.

Solids contents for these campaigns were lower than anticipated. Part of the reason for the poorer results was the concentration of the operating staff on producing sludge cake at a high rate, unfortunately at the expense of cake solids content. Consequently, the solids contents were lower than anticipated. In Campaign IV, with Jersey City polymer-conditioned primary sludge, higher pressures were used (up to 827 k Pa - 120 psig) and the squeezing period was extended to about 47 minutes. Sludge cake averaged 51.0 percent solids.

These results point up an important consideration. It is important to have a properly designed dewatering system with adequate capacity if thermal conversion processes are to be economically attractive. Diaphragm filter presses can produce sludge cake at higher than design rates if cycle times are shortened, but lower cake solids will result.

Operating Modes and Campaign Descriptions

In the course of the investigation, four thermal conversion modes were used, one with greater than stoichiometric oxygen supply (TC-GSOS) and three with sub-stoichiometric oxygen supply (TC-SSOS). The presentation will be made simpler if these modes are explained in advance. The modes are:

TC-GSOS: Incineration (INC)

TC-SSOS: Low temperature char (LTC)
: High temperature char (HTC)
: Carbon burned ash (CBA)

INC: Incineration is the normal operating mode for sludge disposal in an MHF. Air in substantial excess is introduced on the lower hearths and is still in excess in the exhaust gas leaving the MHF. Sludge dries on the upper hearths, volatile combustible gases are released and burn in the overhead space in the middle hearths, and the fixed carbon is burned out of the solids as they proceed to the lower hearths. Normally there is a substantial countercurrent effect. Ash is cooled by incoming air and exiting gases are cooled as they evaporate moisture from the sludge. Cooling of the exiting gas is desirable because there is excess air present. Combustion could take place on the upper hearths if the gas were hot and could continue out of the furnace into the duct work. Unfortunately, volatile hydrocarbons released into the cool exiting gas do not combust, and excessive hydrocarbon levels can result.

LTC, HTC: Both of these operating modes produce char. LTC utilizes a lower average temperature in the furnace so the char contains more VOL and FC. An arbitrarily selected maximum temperature is 650 °C. HTC utilizes temperatures up to 870 °C. For both methods, it is important not to have high oxygen concentrations near the solid phase, because this could cause combustion at the surface of the solids. Low oxygen concentrations should prevail on all hearths, The difference between the LTC and HTC mode is indistinct --there is actually a continuum of conditions with the proportion of VOL and FC relative to inerts in the ash depending on the temperature level. Because there is a deficiency of oxygen for combustion, the gases leaving the furnace will contain unburned gaseous substances.

CBA: Like LTC and HTC, this mode uses a sub-stoichiometric amount of air; that is, less oxygen is added than is needed to completely burn the sludge. However, on the lower hearths, a local excess of oxygen is used in order to burn out the residual VOL and FC in the ash. In the upper hearths, the gas streams will contain combustible gases such as CO, possibly H_2, and low molecular weight hydrocarbons.

A general description of the experimental campaigns, their objectives and findings follows. Table 2, which will be used in discussions later on, lists the significant runs in the various campaigns.

CAMPAIGN I: Coney Island digested sludge was polymer conditioned and filter pressed. Average solids content was 32-35%. No cake breaker was used. Cake frequently came through furnace in unburned lumps; consequently, a cake breaker was used in all later campaigns. This campaign was primarily a learning operation. For example, in Table 2, four of the runs in Campaign I used less then 10 percent of the theoretical air needed for combustion. Excessive quantities of natural gas were used during these runs for heating the MHF.

CAMPAIGN II (A and B): Coney Island digested sludge was polymer conditioned and dewatered with the diaphragm press to about 42% solids. The II A portion of this campaign was conducted in a winter weather emergency and natural gas could not be used. The furnace was preheated by burning wood in it. Fortunately, the sludge cake was high in solids and autogeneous operation was achieved. The furnace run summary sheet for Run II A 3 is shown in Figure 3. Inlet air flow to each hearth was modulated to keep as nearly uniform temperature as possible on the

TABLE 2

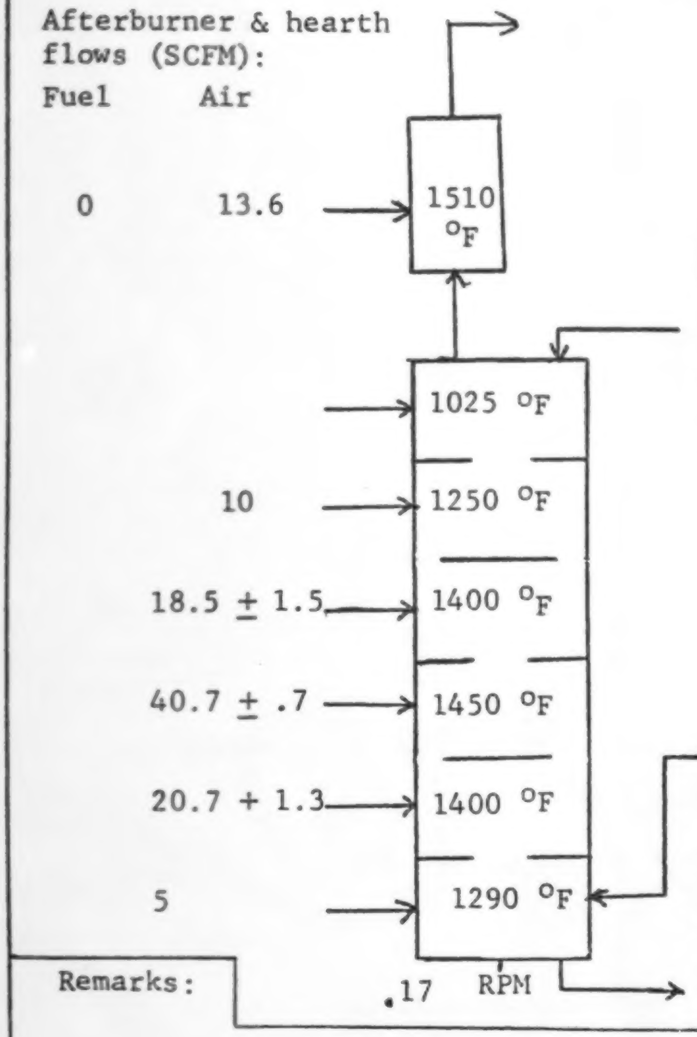
Summary of Pertinent Runs Made in Campaigns I, II, and III

Campaign	Run No.	Run Duration (HR)	Net Air Flow to Furnace (SCFM)	Theoretical Air Needed to Combust Sludge (%)	VC and Vol. in Residue (%)	T av.[5] (°C)	Operating Modes
I	1		107	111.7	3.15		
	3		(-6)	-6	23.5		
	6		8	8.4	28.0		
	7[1]		7	7.3	24.3		
	8		(-10)	-10.4	4.5		
	9		127	133.0	12.6		
	10[2]		128	134.0	0.7		
	12[2]		101	105.0	2.4		
II A	1[1]	6	69.4	45.4[3]	19.1	654	HTC
	2[1]	5	111.9	73.3[3]	38.4	514	LTC
	3[1]	5.75	94.9	62.1[3]	20.3	705	HTC
II B	1R[1]	5	225.8	170.8	2.2	783	INC
	2.1	6	91.2	69.0[3]	35.0	644	LTC,S-D[4]
	2.2	7	116.2	87.9[3]	23.9	687	HTC
III	1[1]	5.5	70.7	58.5[3]	31.0	538	LTC,S-D[4]
	2[1]	12.3	72.5	60.0[3]	19.6	648	HTC,S-D[4]
	3	4	106.3	88.0	7.1	679	CBA
	4	2.5	166.1	137.5	4.5	639	INC
	5[1]	5	217.0	179.5	5.5	602	INC
	6[1]	5	146.7	121.4	9.7	684	INC,LowO_2
	7	2	117.5	97.3	9.4	666	CBA
	8	4.75	116.9	96.8	13.6	684	CBA

1 - Runs that were 5 hours or longer and had no significant process upset.
2 - Steam injection
3 - Data for these runs were plotted in Figure 6
4 - S-D means split draft.
5 - T av. is the average of the six hearth temperatures.

Furnace run summary sheet, Campaign II, Part A

Steady state 3 Date 2-3 & 4, 1977
Time 21:30 of 2/3/77 to
03:15 of 2/4/77


Afterburner & hearth flows (SCFM):
Fuel
Air
0
13.6
1510 °F
1025 °F
10
1250 °F
18.5 ± 1.5
1400 °F
40.7 ± .7
1450 °F
20.7 + 1.3
1400 °F
5
1290 °F
.17 RPM

Stack gas: DSCFM
%CO$_2$ %O$_2$ %CO
Grains/DSCF

Feed analysis/flow:

Total	250	lb/hr.
Water	156.20	lb/hr.
Solids	93.80	lb/hr.
Ash	16.49	lb/hr.
Vol.	64.93	lb/hr.
F.C.	12.38	lb/hr.
H.V.	9039	BTU/#(dry)

Steam 0 lb/hr.

Product analysis/flow

Total	17.42	lb/hr.
Ash	13.88	lb/hr.
Vol.	.623	lb/hr.
F.C.	2.913	lb/hr.
H.V.	3795	BTU/#(dry)

Remarks:

Pyrolysis
Andersens taken at 22:37 & 01:48

FIGURE 3: SUMMARY FOR RUN II A 3

central hearths. Note that the gases leaving the furnace increase in temperature 485 °F (270 °C) when air is added to the afterburner.

In the B portion of Campaign II, the "split-draft" technique was utilized. Instead of withdrawing all of the exhaust gases from the top hearth, the damper in the line connecting Hearth 6 (the lowest hearth) to the manifold leading to the afterburner was partially opened. Splitting the "draft" between the top and bottom hearths leads some of the combustion gases downward. Because there is an inflow of gases low in oxygen into the lower hearths, introduction of air at the lower hearths is less likely to cause combustion at the solid surface in these hearths. Additionally, if the gases contain incompletely burned gases, heat can be generated in the lower hearths by addition of air. Unfortunately, it was not possible to measure the amount of the gases drawn from the top and bottom hearth. Judging from the position of the dampers, the flow from the top hearth was about 3 to 4 times the flow from the bottom hearth in split draft operation.

CAMPAIGN III: Feed was Rahway, N. J., raw sludge, polymer conditioned and dewatered on the diaphragm press. Average solids content was 31.5%. This run was largely devoted to developing comparative LTC, HTC, and INC conditions for air pollution studies. Split drafting was used in the LTC and HTC runs. A few CBA runs were made. Figure 4 shows a run summary sheet for Run III.3. Relatively high flow of air to the lower hearths burned the VOL and FC out of the residue. Note the substantial temperature increase from the top hearth to the afterburner. The small amount of natural gas (1.2 SCFM) added at the afterburner does not add significantly to the temperature increase. It generates about 1270 kJ/min, which just slightly overcompensates for the afterburner heat losses, which are estimated to be 875 kJ/min at 1600 °F (871 °C).

CAMPAIGN IV: Feed was Jersey City, N. J., raw sludge, polymer conditioned and dewatered on the diaphragm press. Solids content averaged 51.0%. This campaign took place about a year after the other campaigns. Its purpose was to obtain definitive information on air pollutional discharges and the effect of the various operating modes on these discharges.

Residence Time in the MHF

During the course of a run in Campaign I, a batch of sludge cake which had been conditioned with ferric chloride and lime was introduced into the MHF with no attempt to blend it beforehand with the preceding polymer-conditioned batch. This provided an excellent opportunity to determine the residence time in the MHF by determining iron and calcium content of the exiting ash. Rabble arm speed during this run (Run I.12) was 0.17 RPM.

Results of the experiment obtained with calcium are presented in Figure 5. The ordinate shows the relative departure that still exists between C and its ultimate value C_F. Results were quite similar for iron but the experimental points more scattered. Residence time of 1.57 hours, assuming plug flow, was calculated on the basis of an average depth of 1.5 inches on

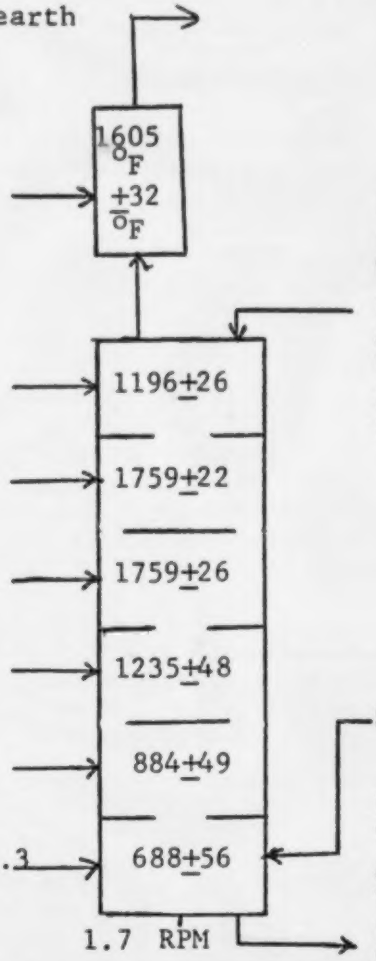
Furnace run summary sheet, Campaign III

Steady state 3 Date 6/23/77
Time 3-7:00

Stack gas: DSCFM
%CO_2 %O_2 %CO
Grains/DSCF

Afterburner & hearth flows (SCFM):

Fuel	Air	
1.2+.3	126	1605 °F ±32 °F
0	0	1196±26
0	0	1759±22
0.88±.11	18±.5	1759±26
1±.05	31±2	1235±48
0	38±.5	884±49
0	37.4±1.3	688±56



Feed analysis/flow:

Total	245	lb/hr.
Water	164.64	lb/hr.
Solids	80.36	lb/hr.
Ash	18.88	lb/hr.
Vol.	54.08	lb/hr.
F.C.	7.47	lb/hr.
H.V.	10,109	BTU/#(dry)

Steam 0 lb/hr.

Product analysis/flow

Total	20.7	lb/hr.
Ash	19.23	lb/hr.
Vol.	1.34	lb/hr.
F.C.	0.13	lb/hr.
H.V.	573	BTU/#(dry)

1.7 RPM

Remarks: Reducing atmosphere in the devolatilizing hearths and excess oxygen in the carbon burning hearths.

FIGURE 4: SUMMARY FOR RUN III 3

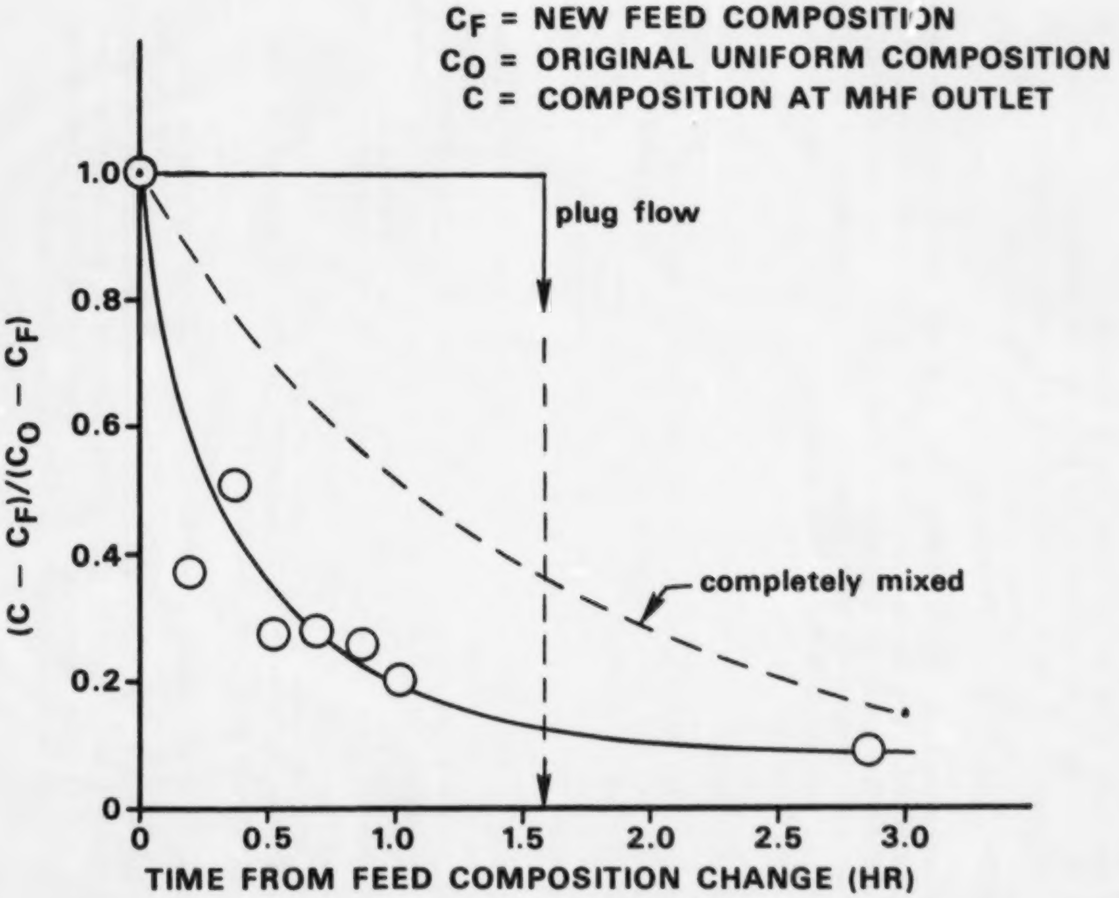


FIGURE 5 EFFECT OF A STEP CHANGE IN FEED COMPOSITION ON OUTLET CONCENTRATION

the available hearth area and an average ash density of 0.32 g/cm^3. Passage of the sludge ash through the unit was much faster than the plug flow residence time would indicate. More rapid passage was expected than predicted by the plug flow model because there is always a "dead bed" residue on the hearths of a MHF that does not move and slowly mixes with the surface materials. Passage, however, is even faster than is indicated by rabble arm speed (6 min per revolution or 0.6 hour for six hearths). Consideration of the geometry of the MHF, particularly the location of dropholes on the alternating "in" and "out" hearths, indicate that shortcircuiting of sludge or ash to lower hearths could occur. This would account for the rapid appearance of the tracer in the exiting stream.

Residue analyses from INC runs indicated that VOL and FC residues were greater than expected. The rapid passage from inlet to outlet of some solids is probably the cause of the high values. This effect would doubtlessly be lower with larger diameter furnaces, and burnout would be better. The experiment serves as a reminder that scale-up to full scale equipment must be done judiciously.

Solid Products Produced

As noted earlier, the furnaces can be run in four modes, LTC, HTC, CBA, and INC. The CBA and INC modes produce essentially the same product, a thoroughly burned-out incinerator ash. The HTC mode produces a char with very little volatile material whereas the LTC mode contains more fixed carbon and substantially more volatile material.

A comparison of the feed sludge with products is shown in Table 3 for two operating modes. As Table 2 shows, in the runs corresponding to these char samples, oxygen supply was substantially below the stoichiometric requirement for combustion of the sludge. The temperature shown in Table 3 is the average of the six hearth temperatures. The lower average temperature produces a material higher in both volatiles and fixed carbon than the high temperature runs. Despite significant differences in volatiles and fixed carbon, the chars have similar heats of combustion on an ash-free basis. The LTC chars have heats of combustion on a total mass basis that are as high as RDF and some poor quality coals.

Both the LTC and HTC chars are free-flowing mixtures of powder and granules. Bulk densities are low--substantially lower than for incinerator ash. A char was removed from the MHF and burned to a low VOL and FC content. Samples were removed periodically during the experiment and unsettled density as well as VOL and FC were measured. The following equation expresses the relationship obtained between density and the sum of VOL and FC:

$$\rho = 3.2 \times 10^{-4} X^2 - 0.02X + 0.575$$

where X = sum of FC and VOL
ρ = density (g/cm^3)

TABLE 3

Comparison of Char Produced in LTC and HTC Operating Modes with Feed Sludge

Material Characteristics

Run	Material Source	%Ash	%Vol	%FC	Heat of Combustion (kcal/kg)		Average Temperature[1] (°C)	Operating Mode
					Actual	Ash-Free		
II A2	Feed	16.8	71.6	11.6	5205	6256	514	LTC
	Residue	61.6	9.1	29.3	3048	7939		
II A3	Feed	17.6	69.2	13.2	5021	6094	705	HTC
	Residue	79.7	3.6	16.7	1618	7972		
III 1	Feed	24.3	66.7	9.0	5806	7672	538	LTC
	Residue	69.0	11.7	19.3	2313	7461		
III 2	Feed	25.1	65.9	9.0	5659	7555	648	HTC
	Residue	80.4	3.5	16.1	1542	7867		

1 - Average of the six hearth temperatures

Typical densities (calculated from this equation) are:

Product	% FC + VOL	(g/cm^3)
LTC char	25	0.28
HTC char	12.5	0.32
incinerator ash	2	0.55

The properties of the chars, such as adsorbent power or degree of water solubility have not yet been investigated.

Correlation of Char Content versus Furnace Conditions

For a given sludge, under conditions that the bed of solids in the hearth is not exposed to a high oxygen concentration, the rate of loss of VOL and FC would be expected to primarily depend on the temperature-time relationship as the solids pass through the furnace. When the bed is not incandescent from combustion on its surface, the temperature of the solids is probably reasonably well represented by the temperature of the surrounding refractories, since the heat received by surfaces at these temperature levels is primarily by radiation. The only temperature measurements available are readings of thermocouples in the gas phase above the hearth, which are exposed to convection and radiation. These readings would be intermediate between gas and surface temperatures and can be used as an approximation to the surface temperature.

Figure 6 shows the percent FC and VOL in the residues that have passed through the furnace plotted against the average of the six hearth temperatures. The runs selected for this figure are indicated in Table 2. Only LTC and HTC runs where less than 90 percent of the theoretical air was used are included. The correlation is reasonably good when it is considered that the temperature profiles were not uniform from run to run and conditions were not held constant very long. Correlation of FC and VOL content against the sum of the fourth power of the absolute temperatures on each hearth ($\Sigma = T_1^4 + T_2^4 \ldots + T_6^4$) was attempted because at elevated temperatures heat transfer rate should be proportional to the fourth power of the absolute temperature. However, no correlation was achieved. Evidently, the temperature level experienced by the char thus is more important than the average rate of radiative heat transfer on the hearths.

Material and Heat Balances

Several serious problems were encountered in the investigation in attemptingto close heat and material balances. Leaks were present in the system. There was some outleakage from the furnace and some inleakage at the afterburner. Gas analyses at the furnace exits and the afterburner exit proved unsuccessful, due to difficulties that occurred in transporting the samples offsite to the gas chromatograph. Calculations of mass and energy flows were made from inputs and reasonable assumptions for gas composition.

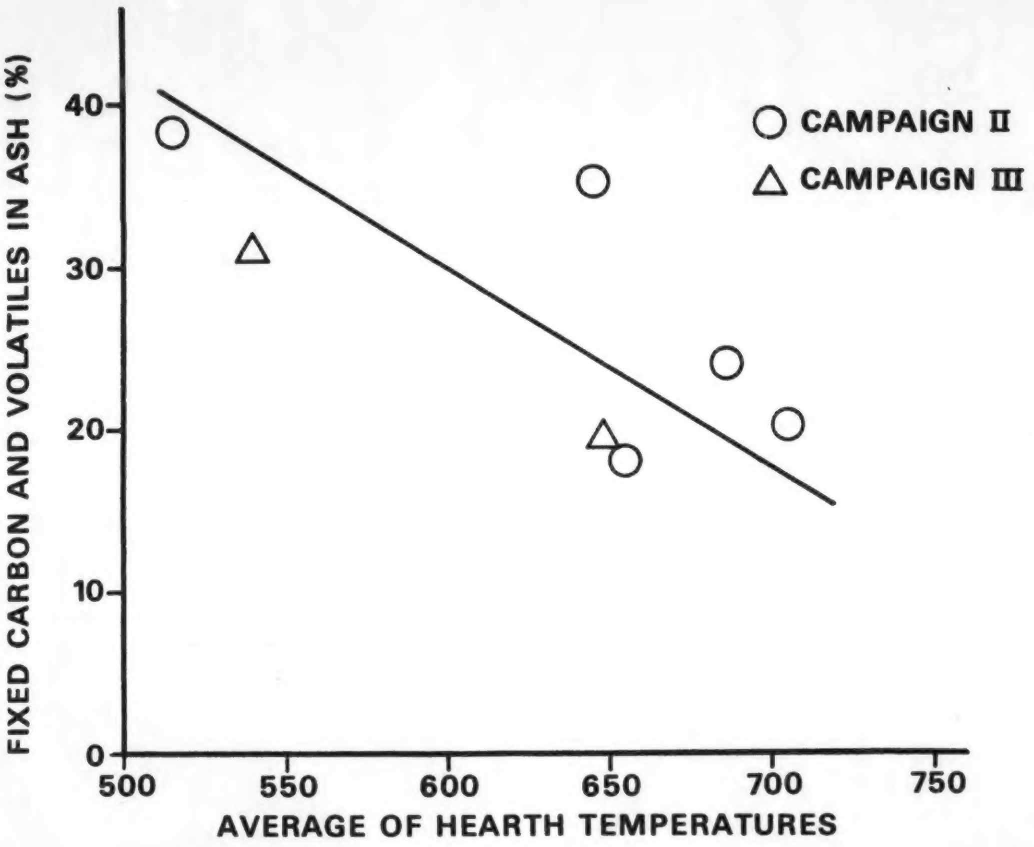


FIGURE 6 EFFECT OF TEMPERATURE ON FIXED CARBON AND VOLATILES CONTENT OF ASH UNDER REDUCING CONDITIONS

Combustion in the Afterburner

There are two important questions about afterburner performance-does the gas have a sufficiently high heat of combustion to ignite and burn at an adequately high rate, and are the hydrocarbon gases reduced in concentration to meet air pollution requirements?

Because gas analyses are not available for the gases exiting from the furnace, the heat of combustion cannot be determined with a high degree of confidence. However, afterburner temperature and the top hearth temperature are known. Consequently, when split-draft is not being used and natural gas usage at the afterburner is small, it is possible to determine whether the discharge gases are igniting and whether there is enough fuel value to cause a substantial temperature increase. Four runs for which these conditions apply are shown below:

Run	Operating Mode	Top Hearth (°C)	Afterburner (°C)	T(°C)
II A 1	HTC	399	927	528
II A 2	LTC	304	810	506
II A 3	HTC	552	821	279
III 3	CBA	647	874	127

In Run III 3, an amount of natural gas was used that approximately offset the heat loss from the afterburner.

The above data indicate that the offgases do ignite and produce temperatures high enough for practicable recovery and very likely high enough to completely oxidize all unburned hydrocarbons. Unfortunately, the afterburner is substantially oversized. Gases exit from the afterburner at an actual rate on the order of 0.4 m^3 per second (dry gas basis), giving a residence time of 5.0 seconds. Commercial scale afterburners have shorter residence times (but probably better mixing). It is likely that the proof of the operability of the afterburner will have to await more carefully scaled tests.

The second question posed above must also await further tests. Experiments using the oversized afterburner at the Nichols pilot plant would not produce conclusive evidence of performance of afterburners with more practical residence times.

Losses of Metals

As noted earlier, difficulty was experienced in closing the material and energy balances because gas analyses were lacking. The material balance problem is simple for substances that remain in the solid phase, particularly the metals. Since feed and residue rates were measured, a material balance is possible for all runs for which particulate samples are available.

When results for Campaign I, II, and III were analyzed, material balances around the MHF and the afterburner showed that losses of lead and cadmium were occurring for the higher temperature runs. Principally to obtain more reliable data and also to determine whether certain operating modes gave lower losses, Campaign IV was run.

A complete analysis of the results of Campaign IV has not been made, so the present discussion must necessarily be superficial. However, the results support the findings of the first three campaigns--the volatile metals, particularly lead and cadmium, show poor material balances for the high temperature processes. Output is always lower than input.

Figure 7 shows the results obtained for INC, LTC, and HTC modes. Consider the HTC mode for cadmium. The data show that the sum of the cadmium in the residue and in the unabated particulates amounts to only half of the total cadmium. The remainder is not accounted for. The amount of cadmium in the unabated particulates is of concern, but air pollution control equipment will capture most of this. The amount not accounted for is of serious concern. If our data are correct, this material has escaped capture by a filter designed to remove particles down to 0.1 micrometer in size. Our pollution control equipment may not remove and recover very much of the "not accounted for" fraction. It should be noted that Takeda and Hiraoka (8) have observed similar metal losses in pyrolysis or combustion of sewage sludge.

DISCUSSION AND CONCLUSIONS

The New Operating Modes and Products

The experiments conducted have demonstrated the feasibility of operating a multiple hearth furnace in the CBA, LTC, and HTC modes, to produce a burnable gas at the top of a multiple hearth furnace. The system presents substantial advantage when afterburning is needed. Heat recovery is feasible and no additional fuel is needed for afterburning. Information was not collected on production rates for these new modes of operation. However, Nichols' staff suggest that LTC has the highest rate, and HTC is next. INC has a rate slightly higher than CBA.

The new products, LTC and HTC char, have interesting potential. They could possibly serve as adsorbents for wastewater treatment. LTC char has a heat of combustion, equivalent to some low quality coals, and could serve as a fuel. The LTC char should also contain most of the heavy metals which conceivably could be removed or recovered from it.

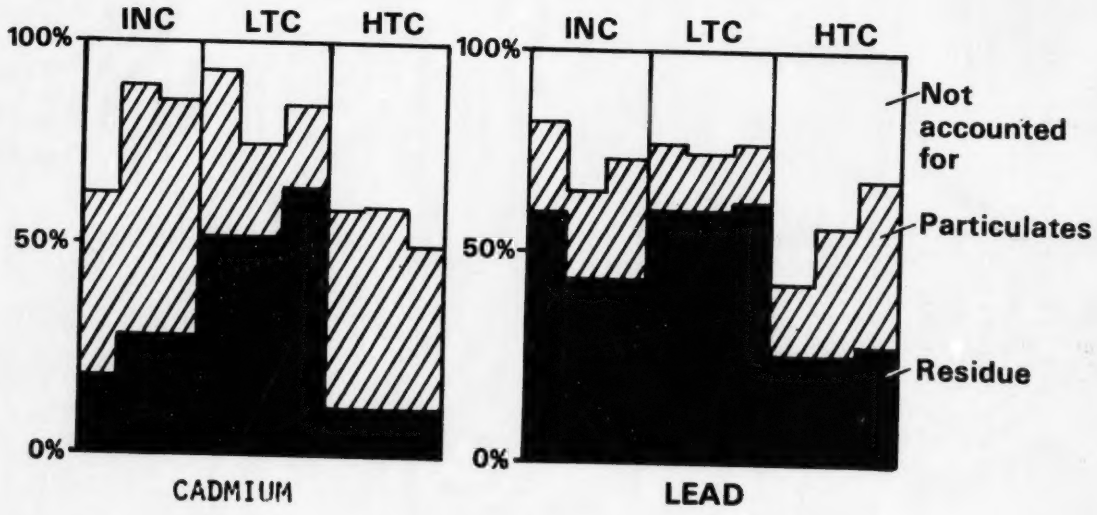


FIGURE 7 MATERIAL BALANCE FOR CADMIUM AND LEAD FOR INC, LTC, AND HTC MODES, CAMPAIGN 4

Losses of Metals

Previous studies by the EPA's Municipal Environmental Research Laboratory (9) have indicated significant amounts of lead and cadmium in the particulates from sludge incinerators that escape collection. Nevertheless, the quantities are not sufficient to seriously threaten environmental quality. If unaccounted for quantities are included, this may no longer be true. It is imperative that a program be commenced soon to determine the true extent of losses (after particulate abatement) from sludge incineration facilities.

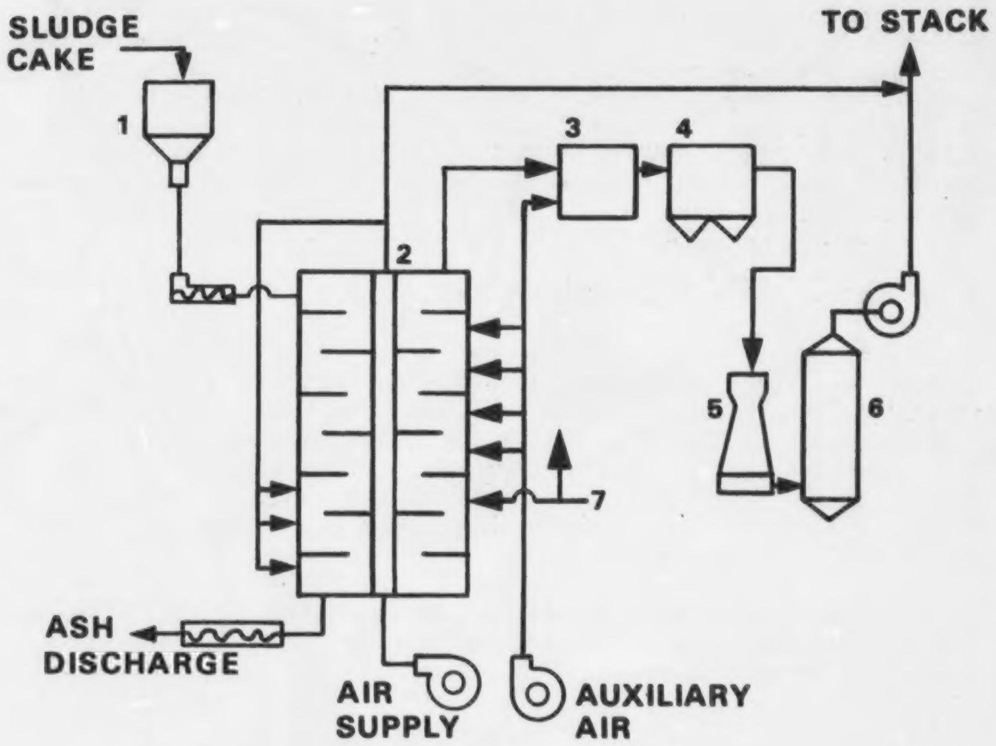
DEVELOPMENTS AND PLANS

A most significant development is the construction of a TC-SSOS facility at Arlington, Virginia. The unit which is outlined schematically in Figure 8 has been designed by Alexander Potter Associates with the assistance of Nichols Engineering and Research Corporation. The unit is not equipped for split-draft, but is designed to operate in the CBA mode. Heat recovery with a waste heat boiler and power generation by means of a steam turbine is planned.

EPA's research plans will concentrate on evaluation of facilities such as the Arlington plant, and on the problem of metal losses during the various thermal conversion modes.

ACKNOWLEDGEMENT

The work was an integrated effort of the Interstate Sanitation Commission, Nichols Research and Engineering Corporation, and the U. S. EPA. Thanks are extended to Dr. Alan Mytelka of the ISC, Mr. Charles Von Dreusche of Nichols, Mr. Walter Lobo, consultant to Nichols, and Mr. Howard Wall, EPA project officer, for their leadership of this joint effort.


SLUDGE CAKE
TO STACK
1
2
3
4
5
6
7
ASH DISCHARGE
AIR SUPPLY
AUXILIARY AIR

1 Sludge cake storage hopper
2 Eight-hearth MHF
3 Afterburner
4 Waste heat boiler
5 Venturi Scrubber
6 Tray Scrubber
7 Fuel and combustion air to hearths and to afterburner

FIGURE 8 SAC UNIT UNDER CONSTRUCTION AT ARLINGTON, VIRGINIA - CBA MODE
(SAC - STARVED AIR COMBUSTION)

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OCCURRENCE AND REMOVAL OF TOXICS IN MUNICIPAL WASTEWATER TREATMENT FACILITIES*

John J. Convery, Jesse M. Cohen, Dolloff F. Bishop**

INTRODUCTION

Some 20 years ago there was a growing concern by the predecessor organizations of the U.S. Environmental Protection Agency that synthetic organics (and certain naturally prevalent compounds) were contaminating the environment - air, water and soil. Thus it was that investigations on controlling these compounds have historically been an integral part of the research program. Still, some excellent research was accomplished; for example, water sources and drinking water were shown to contain substantial amounts of organic matter as measured, in a gross way, by the carbon chloroform extract (CCE) and by the carbon alcohol extract (CAE). By tedious wet- chemical methods, specific organics were identified using such techniques as carbon adsorption for concentrating the organics, infra-red and UV spectroscopy for compound identification and gas chromatography when that instrument became available.

Such crude (in retrospect) techniques produced results slowly, but the information gained in this way was sufficient to confirm the widespread prevalence of synthetic organics in the environment and to arouse concern about their effects. Confirmation of these early data proceeded rapidly when analytical methodology and instrumentation was developed to cope with the problem of qualitatively and quantitatively determining the vast array of organic and inorganic compounds that are produced, discarded and environmentally dispersed by our modern society. Chemical sales in the United States in 1978 involved some 70,000 different substances, and in recent years, almost 1,000 new chemicals have been introduced into commercial production annually.

By a series of legislative acts, the Congress of the United States charged EPA and other agencies with the responsibility of safeguarding the public as well as the environment from the hazards of toxic chemicals. The objective of this paper is to review, briefly, the legislative mandates, the impact that these acts have on a segment of the environment, - the municipal wastewater treatment plant - and finally a description of the research which responds to the problem of toxic compounds in municipal wastewaters.

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* Paper for presentation at the Seventh Joint United States/Japan Conference in Tokyo, Japan - May 1980.

**Director, Wastewater Research Division, Municipal Environmental Research Laboratory, USEPA; Chief, Physical-Chemical Treatment Section, WRD; Chief, Technology Development Support Branch, WRD.

REGULATORY FRAMEWORK FOR URBAN TOXICS MANAGEMENT

The need for data on the occurrence and removal of toxics in municipal wastewater treatment facilities is derived from the informational requirements of environmental legislation, the consent decree agreement (1) and the implementation requirements of federal, state and local regulatory agencies. The primary sources of toxics, their routes of transport in an urban environment and applicable environmental laws to control toxics are shown schematically in Fig. 1. Summaries of the important laws for toxics management are presented in Appendices A and B. Toxics control management requires a systematic approach of limiting environmental exposures through the application of discharge restrictions on important sources which are consistent with control technology capabilities, economic considerations, environmental management objectives as well as environmental quality criteria.

An essential element in EPA's urban toxic control strategy is the use of industrial pretreatment regulations. The intentions of the Agency in applying these regulations are to protect the operational stability of the municipal wastewater treatment facilities, limit the pass-through of toxics to the receiving waters and, conceptually important, to maintain the quality of municipal sludges at levels suitable for environmentally safe disposal, including reclamation and reuse, if possible.

The pretreatment regulations establish the means for applying and enforcing technology-based standards for 21 major industrial categories (including 294 sub-categories) to control some 65 classes of potentially toxic industrial pollutants (defined as 129 specific compounds). Separate regulations are being developed and promulgated for each industrial category which will define national standards based on a determination of the "best available control technology which is economically achievable" (BATEA). The regulated industries and toxics (priority pollutants) to be controlled are listed in Tables 1 and 2, respectively. Candidate BATEA technologies are being evaluated in terms of their toxics removal capabilities. Operationally, monitoring of the BATEA pretreatment facilities will not include specific analyses of all or even of many of the priority pollutants because of costs and analytical service availability considerations. Treatment facility performance will be monitored using more traditional parameters which will document the level of operational stability, and, it is hoped, will indicate or infer a level of control of the priority pollutants.

Pretreatment programs will be required for publicly owned treatment systems with a total design flow greater than five million gallons per day which receive industrial wastes which are subject to pretreatment regulations. EPA estimates that approximately 568 sewage authorities will be requiring pretreatment from about 40,000 industrial sources.

Primary responsibility for implementing the pretreatment program resides either with the local authorities with back-up by EPA or with the 32 states

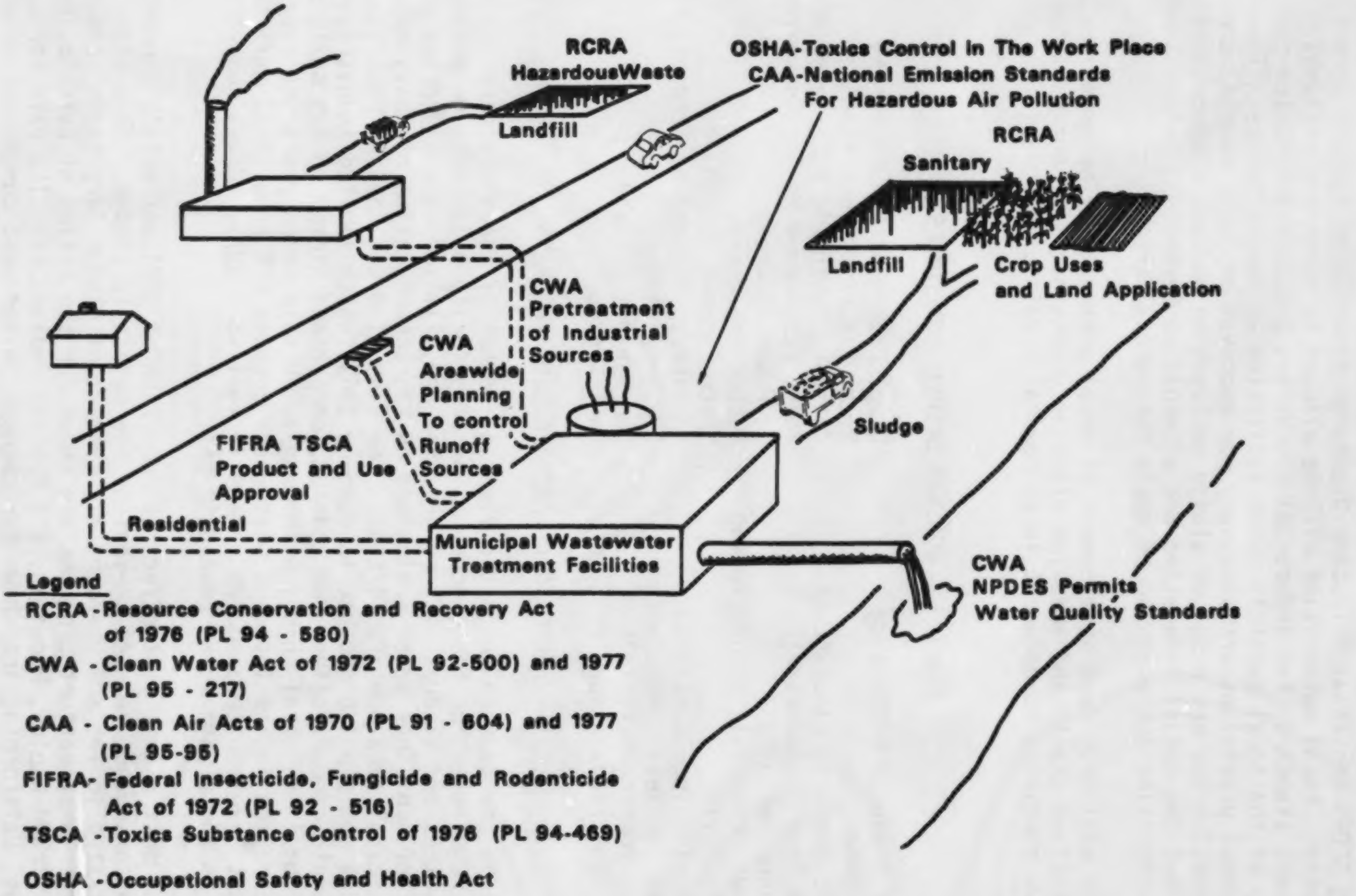
RCRA
HazardousWaste
Landfill
OSHA-Toxics Control In The Work Place
CAA-National Emission Standards
For Hazardous Air Pollution
RCRA
Sanitary
Landfill
Crop Uses
and Land Application
CWA
Pretreatment
of Industrial
Sources
CWA
Areawide
Planning
To control
Runoff
Sources
Sludge
FIFRA TSCA
Product and Use
Approval
Residential
Municipal Wastewater
Treatment Facilities
CWA
NPDES Permits
Water Quality Standards
Legend
RCRA - Resource Conservation and Recovery Act of 1976 (PL 94 - 580)
CWA - Clean Water Act of 1972 (PL 92-500) and 1977 (PL 95 - 217)
CAA - Clean Air Acts of 1970 (PL 91 - 604) and 1977 (PL 95-95)
FIFRA - Federal Insecticide, Fungicide and Rodenticide Act of 1972 (PL 92 - 516)
TSCA - Toxics Substance Control of 1976 (PL 94-469)
OSHA - Occupational Safety and Health Act

Figure 1. Toxics Transport In An Urban Environment And Applicable Environmental Legislation

having NPDES (National Pollution Discharge Elimination System) permit authority. Local authorities will be allowed to relax the national pretreatment standards for industrial discharges based on documented capability of the local publicly owned facilities to remove toxics. Two additional provisions are necessary for approval of local removal credits. The practice may not preclude sludge reclamation and reuse opportunities nor limit the use of innovative and alternative technology. Approximately 360 authorities are expected to apply for removal credits.

EPA will help fund development of local pretreatment programs through construction grants and areawide planning grants but will not fund the studies necessary to document local removal credits.

TABLE 1. CONSENT DECREE INDUSTRIES

Timber Products Processing
Auto, Other Laundries
Organic Chemical Manufacturing
Iron & Steel Manufacturing
Petroleum Refining
Inorganic Chemicals Manufacturing
Textile Mills
Leather Tanning, Finishing
Non-Ferrous Metals Manufacturing
Paving, Roofing Materials
Paint, Ink Formulation; Printing
Soap, Detergent Manufacturing
Steam Electric Power Plants
Plastic, Synthetic Materials, Mfg.
Pulp, Paperboard Mills; Converted Paper Products
Rubber Processing
Miscellaneous Chemicals
Machinery, Mechanical Products Mfg.
Electroplating
Ore Mining, Dressing
Coal Mining

EFFECTS OF PRETREATMENT PROGRAMS

It is too soon to judge the eventual impact of industrial pretreatment BATEA regulations on the concentration of priority pollutants in municipal wastewaters and sludges. There are, however, limited case histories of the impact of municipal sewer ordinances on the concentration of heavy metals on municipal treatment facilities. The City of Grand Rapids, Michigan, passed a municipal ordinance in January 1969, which limited industrial effluent metals concentrations for cadmium, copper and nickel to 1.5 mg/l; zinc to 6.0 mg/l and total chromium to 2 mg/l. Within seven years the total metal concentration in the influent wastewater dropped 87% from 12-13 mg/l to 2 mg/l. Effluent levels decreased 92% from 9-10 mg/l to 1 mg/l. Metal concentrations in the sludge were reduced 58-66%.

The most significant effect of reduction of influent metals concentration is the consequent reduced content of the metals in sludge. This effect is illustrated in Table 3 where metals content of sludge (dry basis) before and after pretreatment programs are shown for the cities of Buffalo, New York; Grand Rapids, Michigan; and Muncie, Indiana.(2) The data for Los Angeles, California, are shown for changes in influent concentration of metals resulting from industrial pretreatment programs. (3)

TABLE 2 PRIORITY POLLUTANTS

Compound Name

1. *acenaphthene
2. *acrolein
3. *acrylonitrile
4. *benzene
5. *benzidine
6. *carbon tetrachloride (tetrachloromethane)

*Chlorinated benzenes (other than dichlorobenzenes)

7. chlorobenzene
8. 1,2,4-trichlorobenzene
9. hexachlorobenzene

*Chlorinated ethanes (including 1,2-dichloroethane, 1,1,1-trichloroethane and hexachloroethane)

10. 1,2-dichloroethane
11. 1,1,1-trichloroethane
12. hexachloroethane
13. 1,1-dichloroethane
14. 1,1,2-trichloroethane
15. 1,1,2,2-tetrachloroethane
16. chloroethene

*Chloroalkyl ethers (chloromethyl, chloroethyl and mixed ethers)

17. bis(chloromethyl) ether
18. bis(2-chloroethyl) ether
19. 2-chloroethyl vinyl ether (mixed)

*Chlorinated naphthalene

20. 2-chloronaphthalene

*Chlorinated phenols (other than those listed elsewhere; includes trichlorophenols and chlorinated cresols)

21. 2,4,6-trichlorophenol
22. parachlorometa cresol

*Specific compounds and chemical classes as listed in the consent decree.

TABLE 2 (Contd.)

23. *chloroform (trichloromethane)
24. *2-chlorophenol

*Dichlorobenzenes

25. 1,2-dichlorobenzene
26. 1,3-dichlorobenzene
27. 1,4-dichlorobenzene

*Dichlorobenzidine

28. 3,3-dichlorobenzidine

*Dichloroethylenes (1,1-dichloroethylene and 1,2-dichloroethylene)

29. 1,1-dichloroethylene
30. 1,2-trans-dichloroethylene
31. *2,4-dichlorophenol

*Dichloropropane and dichloropropene

32. 1,2-dichloropropane
33. 1,3-dichloropropylene (1,3-dichloropropene)
34. *2,4-dimethylphenol

*Dinitrotoluene

35. 2,4-dinitrotoluene
36. 2,6-dinitrotoluene
37. *1,2-diphenylhydrazine
38. *ethylbenzene
39. *fluoranthene

*Haloethers (other than those listed elsewhere)

40. 4-chlorophenyl phenyl ether
41. 4-bromophenyl phenyl ether
42. bis(2-chloroisopropyl) ether
43. bis(2-chloroethoxy) methane

*Halomethanes (other than those listed elsewhere)

44. methylene chloride (dichloromethane)
45. methyl chloride (chloromethane)
46. methyl bromide (bromomethane)
47. bromoform (tribromomethane)
48. dichlorobromomethane

TABLE 2 (Contd.)

49. trichlorofluoromethane
50. dichlorodifluoromethane
51. chlorodibromomethane
52. *hexachlorobutadiene
53. *hexachlorocyclopentadiane
54. *isophorone
55. *naphthalene
56. *nitrobenzene
*Nitrophenols (including 2,4-dinitrophenol and dinitrocresol)
57. 2-nitrophenol
58. 4-nitrophenol
59. *2,4-dinitrophenol
60. 4,6-dinitro-o-cresol
*Nitrosamines
61. N-nitrosodimethylamine
62. N-nitrosodiphenylamine
63. N-nitrosodi-n-propylamine
64. *pentachlorophenol
65. *phenol
*Phthalate esters
66. bis(2-ethylhexyl) phthalate
67. butyl benzyl phthalate
68. di-n-butyl phthalate
69. di-n-octyl phthalate
70. diethyl phthalate
71. dimethyl phthalate
*Polynuclear aromatic hydrocarbons
72. benzo(a)anthracene (1,2-benzanthracene)
73. benzo(a)pyrene (3,4-benzopyrene)
74. 3,4-benzofluoranthene
75. benzo(k)fluoranthane (11,12-benzofluoranthene)
76. chrysene
77. acenaphthylene

TABLE 2 (Contd.)

78. anthracene
79. benzo(ghi)perylene (1,12-benzoperylene)
80. fluorene
81. phenanthrene
82. dibenzo(a,h)anthracene (1,2,5,6-dibenzanthracene)
83. indeno (1,2,3-cd)pyrene (2,3-o-phenylenepyrene)
84. pyrene
85. *tetrachloroethylene
86. *toluene
87. *trichloroethylene
88. *vinyl chloride (chloroethylene)

Pesticides and Metabolites

89. *aldrin
90. *dieldrin
91. *chlordane (technical mixture and metabolites)

*DDT and metabolites

92. 4,4-DDT
93. 4,4-DDE (p,p'-DDE)
94. 4,4-DDD (p,p'-TDE)

*endosulfan and metabolites

95. a-endosulfan-alpha
96. b-endosulfan-beta
97. endosulfan sulfate

*endrin and metabolites

98. endrin
99. endrin aldehyde

*heptachlor and metabolites

100. heptachlor
101. heptachlor epoxide

*hexachlorocyclohexane (all isomers)

102. a-BHC-alpha
103. b-BHC-beta
104. r-BHC (lindane)Gamma

TABLE 2 (Contd.)

105. g-BHC-Delta

*polychlorinated biphenyls (PCB's)

106. PCB-1242 (Arochlor 1242)

107. PCB-1254 (Arochlor 1254)

108. PCB-1221 (Arochlor 1221)

109. PCB-1232 (Arochlor 1232)

110. PCB-1248 (Arochlor 1248)

111. PCB-1260 (Arochlor 1260)

112. PCB-1016 (Arochlor 1016)

113. *Toxaphene

114. *Antimony

115. *Arsenic

116. *Asbestos

117. *Beryllium

118. *Cadmium

119. *Chromium

120. *Copper

121. *Cyanide

122. *lead

123. *Mercury

124. *Nickel

125. *Selenium

126. *Silver

127. *Thallium

TABLE 2 (Contd.)

128. *Zinc

129. **2,3,7,8-tetrachlorodibenzo-p-dioxin (TCDD)

*Specific compounds and chemical classes as listed in the consent decree.

**This compound was specifically listed in the consent decree. Because of the extreme toxicity (TCDD), we are recommending that laboratories not acquire analytical standard for this compound.

TABLE 3. EFFECT OF PRETREATMENT PROGRAMS ON METAL CONTENT OF SLUDGE AND WASTEWATER

		Sludge mg/kg Dry Solids Basis			Wastewater mg/l
Metal	Pretreat. Time	Buffalo(a)	Grand Rapids	Muncie	Los Angeles(b)
Cadmium	Before	100		23	0.037
	After	50		9.5	0.019
Chromium	Before	2540	11000	2000	0.70
	After	1040	2700	675	0.43
Copper	Before	1570	3000	1750	0.45
	After	330	2500	700	0.30
Lead	Before	1800		8500	0.40
	After	605		1000	0.34
Nickel	Before	315	3000	8500*	0.31
	After	115	1700	150	0.21
Zinc	Before	2275	7000	5800	1.55
	After	364	5700	2700	1.17

(a) Projected Values

(b) Change in Influent Concentration from January 1975 - January 1977

* Annual report, 1974 and 1975, from Muncie Sanitary District, reports a value of 430 mg/kg instead of the 8500 reported in the Federal Register (2).

ENVIRONMENTAL QUALITY CONSIDERATIONS

Water Quality Criteria

As part of the consent decree agreement EPA is required to promulgate water quality criteria for 65 specified toxic pollutants. Proposed water quality criteria for the 65 priority pollutants are shown in Table 4. (4) These criteria, when issued in their final form after public comment, may serve as the basis for enforceable standards under the administrative rule-making procedures by either the states or EPA as provided for under Sections 302, 303, or 307(a) of the Clean Water Act. Publication of the criteria per se has no regulatory impact.

Recent Congressional testimony (5) before the House Committee on Public Works and Transportation identified 13 states which have general toxic substance standards based on some fraction of the acute lethal levels as determined by aquatic toxicity tests (i.e., 1/10 of the 96 hour TLm). Fourteen states have included limitations on specific toxic pollutants such as arsenic or cyanide in their water quality standards. A summary of the types of state water quality standards is shown in Table 5. The human health effects water quality criteria relate primarily to exposures through drinking water and for consumption of contaminated aquatic life. Where multiple criteria levels were defined for incremental cancer risk levels to man of 10^{-7} to 10^{-5}, the value shown in Table 4 is the most conservative value (higher concentration). These criteria are not to be confused with finished (treated) drinking water standards which are established under the Safe Drinking Water Act. The finished or treated drinking water standards are established on the basis of human health effects criteria plus consideration of control technology capabilities and economics.

Land Disposal Criteria

An interim final* regulation containing the minimum criteria to provide guidelines for the disposal and utilization of wastewater treatment plant sludge was published in September 1979. As proposed, any owner or operator of a publicly owned treatment works must comply with these criteria when disposing of sludge on the land. The essential elements of the proposed criteria are summarized in Appendix C entitled "Criteria for Sludge Application to Land." The criteria are structured to define unacceptable impacts, i.e., those that present a "reasonable probability" of adverse effects on health or the environment. The criteria do not apply to domestic sewage, treated domestic sewage or hazardous wastes. They do apply to solid wastes, which include municipal sludge and septage. The priority pollutants were not specifically included in the criteria to protect groundwater from sludge disposal facilities. Rather, the maximum contaminant levels (MCL) specified in the National Interim Primary Drinking Water Regulations are applicable with the exception of turbidity and man-made radionuclides.

*Interim final means a regulation which is final and enforceable but the Agency is still seeking public comment.

TABLE 4. PROPOSED WATER QUALITY CRITERIA FOR PRIORITY POLLUTANTS

Compound	Freshwater Aquatic Life 24-Hr.Avg µg/l	Freshwater Aquatic Life Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	Saltwater Aquatic Life Ceiling µg/l	Human Health Effects µg/l
1. *Acenaphthene	110	240	7.5	17	$20\mathrm{x}10^{-3}$
2. *Acrolein	1.2	2.7	0.88	2.0	6.5
3. *Acrylonitrile	130	300	130	290	0.084
4. *Benzene	3,100	7,000	920	2,000	15
5. *Benzidine	-	-	-	-	$1.67\mathrm{x}10^{-3}$
6. *Carbon tetrachloride (tetrachloromethane)	620	1,400	2,000	4,600	2.6
*Chlorinated benzenes (other than dichlorobenzenes)					
7. Chlorobenzene					
8. 1,2,4-Trichlorobenzene					
9. Hexachlorobenzene					
*Chlorinated ethanes (including 1,2-dichloroethane, 1,1,1-trichloroethane and hexachloroethane)					
10. 1,2-Dichloroethane					7.0
11. 1,1,1-Trichloroethane					
12. Hexachloroethane					5.9
13. 1,1-Dichloroethane					
14. 1,1,2-Trichloroethane					2.7
15. 1,1,2,2-Tetrachloroethane					1.8
16. Chloroethene					
*Chloralkyl ethers (chloromethyl, chloroethyl and mixed ethers)					
17. Bis(chloromethyl) ether					$0.02\mathrm{x}10^{-3}$
18. Bis(2-chloroethyl) ether					0.42
19. 2-Chloroethyl vinyl ether (mixed)					

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*Specific compounds and chemical classes as listed in the consent decree.

TABLE 4 (Contd.)

Compound	Freshwater Aquatic Life 24-Hr.Avg µg/l	Freshwater Aquatic Life Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	Saltwater Aquatic Life Ceiling µg/l	Human Health Effects µg/l
*Chlorinated naphthalene	29	67	2.8	6.4	.08-3.9[a]
20. 2-Chloronaphthalene					
*Chlorinated phenols (other than those listed elsewhere; includes trichlorophenols and chlorinated cresols)					
21. 2,4,6-Trichlorophenol	52	150	-	-	100
- Tetrachlorophenol					263
22. Parachlorometa cresol					
23. *Chloroform (trichloromethane)	500	1200	620	1400	2.1
24. *2-Chlorophenol	60	180	-	-	0.3
*Dichlorobenzenes (total all isomers)					230
25. 1,2-Dichlorobenzene	44	99	15	34	
26. 1,3-Dichlorobenzene	310	700	22	49	
27. 1,4-Dichlorobenzene	190	440	15	34	
*Dichlorobenzidine					
28. 3,3-Dichlorobenzidine	-	-	-	-	0.1
*Dichloroethylenes (1,1-Dichloroethylene and 1,2-Dichloroethylene)					
29. 1,1-Dichloroethylene	530	1200	1700	3900	1.3
30. 1,2-Trans-dichloroethylene	620	1400	-	-	
31. *2,4-Dichlorophenol	0.4	110	-	-	0.5

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[a] Depends on form (octo-tri)

TABLE 4 (Contd.)

Compound	Freshwater Aquatic Life 24-Hr.Avg. µg/l	Freshwater Aquatic Life Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	Saltwater Aquatic Life Ceiling µg/l	Human Health Effects µg/l
*Dichloropropane and dichloropropene					
32. 1,2-Dichloropropane	920	2,100	400	930	203
33. 1,3-Dichloropropylene (1,3-Dichloropropene)	18	250	5.5	14	0.63
34. *2,4-Dimethylphenol	38	86	-	-	-
*Dinitrotoluene					
35. 2,4-Dinitrotoluene	620	1,400	-	-	0.740
36. 2,6-Dinitrotoluene					
37. *1,2-Diphenylhydrazine	17	38	-	-	0.4
38. *Ethylbenzene	-	-	-	-	1,100
39. *Fluoranthene	250	560	0.30	0.69	200×10^{-3}
*Haloethers (other than those listed elsewhere)					
40. 4-Chlorophenyl ether					
41. 4-Bromophenyl phenyl ether	6.2	14	-	-	
42. Bis(2-chloroisopropyl) ether					11.5
43. Bis(2-chloroethoxy) methane					
*Halomethanes (other than those listed elsewhere)					
44. Methylene chloride (Dichloromethane)	4,000	9,000	1,900	4,400	2
45. Methyl chloride (Chloromethane)	7,000	16,000	3,700	8,400	2
46. Methyl bromide (Bromomethane)	140	320	170	380	2
47. Bromoform (Tribromomethane)	840	1,900	180	420	2
48. Dichlorobromomethane					
49. Trichlorofluoromethane	-	-	-	-	32,000
50. Dichlorodifluoromethane	-	-	-	-	3,000
51. Chlorodibromomethane					

TABLE 4 (Contd.)

	Compound	Freshwater Aquatic Life 24-Hr.Avg. µg/l	Freshwater Aquatic Life Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	Saltwater Aquatic Life Ceiling µg/l	Human Health Fffects µg/l
52.	*Hexachlorobutadiene	-	-	-	-	0.77
53.	*Hexachlorocyclopentadiene	0.39	7.0	-	-	1.0
54.	*Isophorone	2,100	4,700	97	220	460
55.	*Naphthalene	-	-	-	-	143×10^{-3}
56.	*Nitrobenzene	480	1,100	53	120	30
	*Nitrophenols (including 2,4-dinitrophenol and dinitrocresol)					
57.	2-Nitrophenol	2,700	6,200	-	-	
58.	4-Nitrophenol	240	550	53	120	
59.	*2,4-Dinitrophenol	79	180	37	84	68.6
60.	4,6-Dinitro-o-cresol	57	130	-	-	12.8
	*Nitrosamines					
61.	N-nitrosodimethylamine					0.026
62.	N-nitrosodiphenylamine	-	-	-	-	
63.	N-nitrosodi-n-propylamine					
	N-nitrosodiethylamine					0.0092
	N-nitroso-di-n-butylamine					0.013
	N-nitrosopyrolidine					0.11
64.	*Pentachlorophenol	6.2	14	3.7	8.5	140
65.	*Phenol	600	3,400	-	-	3,400
	*Phthalate esters					
66.	Bis(2-ethylhexyl) phthalate					10,000
67.	Butyl benzyl phthalate					
68.	Di-n-butyl phthalate					5,000
69.	Di-n-octyl phthalate					
70.	Diethyl phthalate	-	-	-	-	60,000
71.	Dimethyl phthalate	-	-	-	-	160,000

TABLE 4 (Contd.)

	Compound	Freshwater Aquatic Life 24-Hr.Avg. µg/l	 Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	 Ceiling µg/l	Human Health Effects µg/l
	*Polynuclear aromatic hydrocarbons					9.7x10^{-3}(6)
72.	Benzo(a)anthracene (1,2-benzanthracene)					0.024x10^{-3}
73.	Benzo(a)pyrene (3,4-benzopyrene)					
74.	3,4-Benzofluoranthene					
75.	Benzo(k)fluoranthane (11,12-benzofluoranthene)					
76.	Chrysene					
77.	Acenaphthylene					
78.	Anthracene					
79.	Benzo(ghi)perylene (1,12-benzoperylene)					
80.	Fluorene					
81.	Phenanthrene					
82.	Dibenzo(a,h)anthracene (1,2,5,6-dibenzanthracene)					
83.	Indeno (1,2,3-cd)pyrene (2,3-o-phenylenepyrene)					
84.	Pyrene					
85.	*Tetrachloroethylene	310	700	79	180	2.0
86.	*Toluene	2,300	5,200	100	230	17,400
87.	*Trichloroethylene	1,500	3,400	-	-	21
88.	*Vinyl chloride (chloroethylene)	-	-	-	-	517
	Pesticides and metabolites					4.6x10-5
89.	*Aldrin					4.4x10-8
90.	*Dieldrin					
91.	*Chlordane (technical mixture and metabolites)	0.024	0.36	9.1x10-3	0.18	1.2x10-3
	*DDT and metabolites					0.98x10^{-3}
92.	4,4-DDT					
93.	4,4-DDE (p,p'-DDE)					
94.	4,4-DDD (p,p'-TDE)					

TABLE 4 (Contd.)

	Compound	Freshwater Aquatic Life 24-Hr.Avg. µg/l	Freshwater Aquatic Life Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	Saltwater Aquatic Life Ceiling µg/l	Human Health Effects µg/l
	*Endosulfan and metabolites	0.042	0.49	-	-	100
95.	a-Endosulfan-Alpha					
96.	b-Endosulfan-Beta					
97.	Endosulfan sulfate					
	*Endrin and metabolites	0.0020	0.10	0.0047	0.031	1
98.	Endrin					
99.	Endrin aldehyde					
	*Heptachlor and metabolites					
100.	Heptachlor	0.0015	0.45	0.0036	0.05	2.3×10^{-4}
101.	Heptachlor epoxide					
	*Hexachlorocyclohexane (all isomers)					
102.	a-BHC-Alpha					16×10^{-3}
103.	b-BHC-Beta					28×10^{-3}
104.	r-BHC (lindane)-Gamma					54×10^{-3}
105.	g-BHC-Delta					21×10^{-3}
	*Polychlorinated biphenyls (PCB's)	0.0015	6.2	0.24	.20	2.6×10^{-4}
106.	PCB-1242 (Arochlor 1242)					
107.	PCB-1254 (Arochlor 1254)					
108.	PCB-1221 (Arochlor 1221)					
109.	PCB-1232 (Arochlor 1232)					
110.	PCB-1248 (Arochlor 1248)					
111.	PCB-1260 (Arochlor 1260)					
112.	PCB-1016 (Arochlor 1016)					
113.	*Toxaphene	.007	0.47	0.19	.12	4.7×10^{-4}
114.	*Antimony	120	1,000	-	-	145
115.	*Arsenic	57	130	29	67	0.02
116.	*Asbestos	-	-	-		300,000[b]

[b] 300,000 fibers/liter for 10^{-5} target risk level

TABLE 4 (Contd.)

Compound	Freshwater Aquatic Life 24-Hr.Avg. µg/l	Freshwater Aquatic Life Ceiling µg/l	Saltwater Aquatic Life 24-Hr.Avg. µg/l	Saltwater Aquatic Life Ceiling µg/l	Human Health Effects µg/l
117. *Beryllium	$e^{(1.24 \ln(hard)-6.65)}$	$e^{(1.24 \ln(hard)-1.46)}$	-	-	0.087
118. *Cadmium	$e^{(0.87 \ln(hard)-4.38)}$	$e^{(1.30 \ln(hard)-3.92)}$	1.0	16	10
119. *Chromium					50
120. *Copper	$e^{(0.65 \ln(hard)-1.03)}$	$e^{(0.88 \ln(hard)-1.03)}$	0.79	18	1,000
121. *Cyanide	1.4	38	-	-	200
122. *Lead	$e^{(1.5 \ln(hard)-3.37)}$	$e^{(1.5 \ln(hard)-1.39)}$	-	-	50
123. *Mercury					
124. *Nickel	$e^{(1.02 \ln(hard)-1.02)}$	$e^{(0.47 \ln(hard)+4.19)}$	-	-	133
125. *Selenium	9.7	22	4.4	10	10
126. *Silver	0.009	1.9	0.26	0.58	10
127. *Thallium	-	-	-	-	4
128. *Zinc	$e^{(0.67 \ln(hard)+0.67)}$	$e^{(0.64 \ln(hard)+2.46)}$	-	-	5,000
129. **2,3,7,8-Tetrachlorodibenzo-p-dioxin (TCDD)	-	-	-	-	4.55×10^{-7}

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*Specific compounds and chemical classes as listed in the consent decree

**This compound was specifically listed in the consent decree. Because of the extreme toxicity (TCDD), we are recommending that laboratories not acquire analytical standard for this compound.

TABLE 5. STATE WATER QUALITY STANDARDS

		Specific Water Quality Standard						Effluent Limits		
	Date of Latest W.Q. Std. Revision	D.O. Std.	Ammonia W.Q. STd.	Pesticide Std.	Specific Toxic Sub. Criteria	"Laundry List"	Turbidity Std.	BOD/TSS (1)	Ammonia	Linkage to W.Q. Std.
Alabama	12/77	X					X			X
Arizona	5/76	X			X	X	X			
Arkansas	9/75	X			X		X			
California	2/79	X	X	X	X	X	X			
Colorado	7/78	X	X	X		X	X			
Connecticut	11/77	X								
Delaware	4/75	X	X				X			
Florida	3/79	X	X			X	X			X
Georgia	6/78	X								X
Idaho	6/73	X					X			X
Illinois	5/79	X	X		X			X		X
Indiana	5/78	X			X					
Iowa	3/79	X	X			X				
Kansas	8/78	X	X				X			
Kentucky	7/75	X			X					X
Louisiana	10/77	X					X			
Maine	8/77	X								
Maryland	7/78	X					X			(2)
Massachusetts	2/79	X								
Michigan	12/73	X			X					X
Minnesota	10/73	X	X				X	X		
Mississippi	7/77	X			X					(2)
Missouri	4/75	X								
Montana	9/74	X					X			
Nebraska	10/77	X	X							
Nevada	2/78	X					X	(3)		X
New Hampshire	4/77	X					X			
New Jersey	1/78	X				X	X			
New Mexico	4/77	X								
New York	9/78	X								
North Carolina	12/78	X								(2)
North Dakota	4/77	X	X			X	X			

TABLE 5 (Contd.)

	Specific Water Quality Standard							Effluent Limits		
	Date of Latest W.Q. Std. Revision	D.O. Std.	Ammonia W.Q. STd.	Pesticide Std.	Specific Toxic Sub. Criteria	"Laundry List"	Turbidity Std.	BOD/TSS (1)	Ammonia	Linkage to W.Q. Std.
Ohio	2/78	X	X		X	X				
Oklahoma	3/79	X	X		X		X			
Oregon	4/76	X					X			
Pennsylvania	10/76	X	X			X	X			(2)
Rhode Island	12/77	X					X			
South Carolina	4/77	X								
South Dakota	1/79	X	X		X	X	X			X
Tennessee	12/77	X								(2)
Texas	2/76	X			X			X	X	
Utah	10/78	X	X	X		X	X	X		
Vermont	3/78	X					X			
Virginia	11/74	X								
Washington	12/77	X					X			
West Virginia	4/78	X			X	X				(2)
Wisconsin	6/78	X	X					X	X	
Wyoming	3/79	X					X			

Source: BNA - Environmental Reporter, "State Water Laws" updated as of October 1, 1979

1. Identified are any effluent limits contained in regulations that are more stringent than Secondary Criteria
2. Specific Waste Load Allocation Provisions
3. BOD Water Quality Standards

Hinesly (6) reported on the heavy metal content in runoff and drainage waters from sludge-treated field lysimeter plots. The studies indicated that log normal distributions best characterized the population of metal concentrations with geometric mean concentrations being the most useful for assessing results. The sludge loading rates and heavy metal loading rates were significantly higher than currently recommended loading criteria. The results, shown in Table 6, indicate that copper concentrations increased the most with time and cumulative loading. Cadmium concentrations exceeded the criteria in the 1972-73 period but were largely due to high background levels for the control.

Air Quality Criteria

Specific air quality limits, based on the Occupational Safety and Health Administration (OSHA) air quality standards, have not been promulgated for the practice of sludge disposal to the land. The OSHA standards for 22 of the more volatile priority pollutants are listed in Table 7. (7) These standards are presented in this paper only as a point of reference with which to compare volatile priority pollutant emissions around a municipal treatment facility which may affect the work environment of treatment plant operators. Field measurement of air concentrations over an aerated grit chamber and aeration basin for these volatile priority pollutants are also shown in Table 7.

These data, which are very limited, suggest that air quality above wastewater treatment plant unit operations will generally not be a problem. Forced ventilation of waste treatment facilities will further reduce ambient concentrations. The only potential problem suggested by these data is the concentration of 1,1,2-trichloroethane over the aeration basin.

ANALYTICAL METHODS

For analysis, the priority organics are divided into purgeable and extractable classes. The extractable class is further subdivided into acid, base neutral compounds and into a selected neutral subclass of pesticides and PCB's. The Agency has developed methodology (8) for the measurement of these priority toxic organics based on GC/MS technology. Succinctly, the methodology separates the purgeable priority organics from the environmental sample by purging with inert gas and trapping of the organics on a Tenax and silica gel trap. The organics are then desorbed, identified and quantitated with packed column GC/MS analysis. The methodology separates the extractable organics by extracting with methylene chloride, first at pH 11 and then at pH 2, and then identifies and quantitates the organics in the base neutral and acid extracts by packed column GC/MS analysis.

The basic methodology for purgeable and extractable organics has been extensively applied to four raw municipal wastewaters (9). The initial application of the basic methodology on municipal raw wastewater missed four of the purgeable organics; chloromethane, dichlorodifluoromethane, vinyl chloride and bromomethane. With substitution of charcoal for the

TABLE 6. HEAVY METAL CONTENT IN DRAINAGE FROM SLUDGE APPLIED TO FIELD LYSIMETERS

	Lysimeter Loading				Drainage Water (Control)			Drainage Water (Sludge Amended)		
Year	Annual Sludge Application Rate mT/ha	Zn Kg/ha	Cu (dry	Cd weight)	Zn mg/T	Cu (geo	Cd mean)	Zn mg/l	Cu (geo	Cd mean)
1969	19.33	122	27	5.2						
1970	36.15	312	69	22.2						
1971	104.69	464	114	24.0						
1972	32.19	135	22	5.8	0.019	0.007	0.003	0.037	0.016	0.006
1973	58.75	224	38	6.7	0.016	0.009	0.009	0.038	0.026	0.013
1974	70.73	350	114	21.7	0.016	0.012	0.004	0.033	0.029	0.008
1969	16.37	158	36	7.9						
1970	52.78	427	101	22.6						
1971	57.83	265	69	13.2						
1972	44.38	192	33	7.8	0.021	0.004	0.003	0.021	0.007	0.004
1973	61.08	248	41	6.8	0.015	0.008	0.011	0.018	0.011	0.010
1974	Terminated	-			0.016	0.012	0.004	0.015	0.013	0.005

TABLE 7. ANALYSIS FOR PRIORITY POLLUTANTS IN AIR STREAM FROM GRIT CHAMBER AND AERATION BASIN AT SOUTH BURLINGTON, NC MUNICIPAL TREATMENT PLANT

Priority Pollutants	OSHA STDS[a] (ng/l)	Priority Pollutants - ng/l Grit Chamber	Aeration Basin
Methyl bromide	80,000[b]	ND	ND
Dichloromethane	1,736,000	114 ± 7	117 ± 1
1,1-Dichloroethylene	-	1.5 ± 2.0	ND
Acrylonitrile	-	ND	ND
1,2-*Trans*-dichloroethylene	790,000	188	75 ± 7
Chloroform	240,000[b]	21,963 ± 8,316	3,472 ± 205
1,2-Dichloroethane	-	448 ± 72	320 ± 23
1,1,1-Trichloroethane	1,900,000	4,468 ± 668	487 ± 35
Carbon tetrachloride	62,900	ND	ND
Bromodichloromethane	-	260 ± 34	52 ± 12
1,2-Dichloropropane	350,000	41,850 ± 4,950	35,850 ± 2,350
Benzene	32,000	93 ± 5	126 ± 24
Trichloroethylene	537,000	16,095 ± 1,705	2,931 ± 277
Bromoform	5,000	ND	ND
Tetrachloroethylene	678,000	54,000 ± 9,000	19,300
1,1,2,2-Tetrachloroethane	34,000	ND	ND
Toluene	753,000	31,835 ± 2,665	570
Ethylbenzene	434,000	3,275 ± 275	262
Chlorobenzene	345,000	38	190
1,2-Dichlorobenzene	300,000[b]	ND	19
1,3-Dichlorobenzene	-	88,500	2,700
1,1,2-Trichloroethane	54,600	ND	41,910

a 8-hour time weighted average unless otherwise noted (standards based on T=25°C, P=760 mm Hg)

b maximum - never to be exceeded

silica gel in the Tenax trap and purging at 49°C, the modified approach identified all of the purgeable priority organics and exhibited the best overall recoveries ($\sim$ 90%) for the analysis of priority organics in municipal wastewaters.

The basic methodology (8) does not measure all of the extractable priority pollutants well. N-nitrosodimethylamine does not chromatograph effectively under the conditions of the method and is sufficiently volatile as an extractable organic to result in poor analyses. Hexachlorocyclopentadiene, while successfully determined (10) in some laboratories has been missed by others (9).

The neutral compound, 2-chloroethyl vinyl ether, also was not detected (9) by the basic methodology. Thermal decomposition of 1,2-diphenylhydrazine to azobenzene and N-nitrosodiphenylamine to diphenylamine has also been observed (11). Co-eluting pairs of anthracene-phenanthrene, benzo(a)anthracene-chrysene, and benzo(b)fluoranthene-benzo(k)fluoranthane on the specified packed GC column are not resolved by mass spectroscopy and, therefore, not distinguishable by the methodology. When desired, the use of capillary GC columns (SP-2100 on 30 m wall-coated capillary) in place of the packed column (12) can eliminate the co-elution problem for the three co-elution pairs.

Finally, the bases (benzidines) have been difficult to chromatograph at low concentrations. An alternative HPLC method (13)(14) specifically for benzidines has significantly lowered detection limits. Verification of the benzidines for legal purposes may, however, require a GC/MS procedure.

The detection limits for the organics depend on the sample matrix. Agency estimates on detection limits in wastewaters for the basic methodology are typically 10 μg/l for most of the purgeable and base/neutral priority organics. Agency estimates for most of the acid (phenols) organics in wastewaters are typically 25 ug/l.

Kleopfer, et al., (10), recently completed a statistical evaluation of EPA's basic methodology using data from seven laboratories on both industrial and municipal wastewaters. The statistics (Table 8) indicate an overall average recovery of about 90% of the purgeables and about 80% of the acids (phenols) from both distilled water and wastewater analyses. Significant wastewater matrix effects did not occur for either the purgeables or the acids. Indeed, compared to recoveries in distilled water, the overall average recoveries in the wastewater analyses increased slightly for purgeables and decreased slightly for acids. In the purgeable and acid analyses Kleopfer found that the recoveries for specific organics decreased significantly (purgeables, at the 99% confidence level; acids, at the 95% level) as the volatility of the organics increased.

For base/neutrals, pesticides and PCB's, the study revealed significantly lower average recoveries in the wastewater analyses (68% for base/neutrals, 59% for pesticides and PCB's) compared to those in distilled water (84% for base/neutrals and 78% for pesticides and PCB's). The lower recovery in the wastewater analysis was attributed to increased reactivity of these classes of priority organics. When Kleopfer separated the base/neutral class into more chemically reactive and less chemically reactive groups, the statistical analyses confirmed the greater variability and poorer recoveries in the more reactive grouping.

TABLE 8. RECOVERIES OF PRIORITY POLLUTANTS

	Recoveries (percent)					
	Method Standard*			Sample Spike**		
Priority Pollutant Fraction	P	±	Sp**	P	±	Sp***
Volatile (purgeables)	90	±	13	92	±	15
Acids (phenols)	84	±	13	76	±	19
Base/Neutrals	84	±	25	68	±	21
Pesticides and PCB's	78	±	11	59	±	11

* Method standard refers to recoveries by standard addition to distilled water.

**Sample spike refers to recoveries by standard addition to sample.

***P ± Sp are weighted averages of the data points and are in units of percent recovery ± one standard deviation (Sp).

The quality control data (10) specified in the methodology (8) revealed the quality control limits (± 3 σ) for percent recoveries on individual organics often ranged from zero to several hundred percent. This broad range for some of the organics indicates that either the basic methodology or the analytical performance of the laboratories could be improved. Nevertheless, as an analytical tool for such a wide variety of organics, the methodology with proper quality control is generally satisfactory for the screening analysis of the organics.

Municipal sludges contain sufficient interferences such that the Agency's basic methodology is not successful. The complex samples and those samples where low detection limits (∿1 µg/l) are desired require alternative approaches or additional separation and clean-up procedures. To improve analyses of purgeable organics in sludge samples and to lower their detection limits in all samples, DeWalle and Chian (15) have modified the purge and trap method (Table 9) to include an on-column cryotrap after the Tenax column and ahead of a capillary GC column. The cryotrap, cooled by liquid nitrogen, captures the organics during desorption from the Tenax column.

TABLE 9. ANALYSIS OF PURGEABLE ORGANICS BY CRYOTRAP CAPILLARY GC/MS

5 ml sample

Purging with N_2

Adsorption on Tenax Trap

Desorption at 180°C with back flushing

On column Cryotrapping of desorbed organics with liquid nitrogen

Release of organics and capillary GC/MS analysis
(30 m SE-54 WCOT column)
External standard method for quantitation

The cryotrap is then rapidly warmed to focus and release the organics into the capillary GC. The improved resolution of the capillary column along with the cryotrap is claimed to reduce detection limits of the purge and trap method. Possible future improvements in the purge and trap method to reduce the sludge matrix effects include the use of salts (Na_2SO_4 "salting out") or warming of the sample above ambient temperature to improve the purgeability of the organics.

The large amount of extractable organics in complex samples such as sludges, requires special extraction techniques (16) and separation and clean-up procedures before GC/MS analysis. The principal classes of organic interferences (15)(16) extracted from raw municipal wastewater and sludge samples are:

- Lipids
- Fatty acids
- Saturated hydrocarbons.

In the complex samples, the large amounts of extractable interferences overwhelm both the GC and the mass spectrometer. In order to permit analysis these interferences must, therefore, be reduced in the extract fractions before injection into the GC/MS system.

Three principal conventional approaches are available for this reduction:

- Acid base separation
- Molecular size separation by gel permeation chromatography
- Polarity separation (silica gel chromatography, etc.)

The acid/base separation is the fundamental separation approach behind the Agency's basic methodology (8). In this methodology, base/neutral extraction followed by acid extraction divides the total amount of interferences between acid and base extracts, separates the base/neutrals from the acids and thus reduces the degree of interference in each fraction injected in the GC/MS detector. Acid/base separation, however, may be applied at many points in a separation scheme to remove or separate acid compounds from neutrals or bases in a complex extract.

Molecular size separation is especially effective in removing the lipids and high molecular weight fatty acids and hydrocarbons from the extract. These materials apparently thermally decompose in the GC system and create very complex GC chromatograms. Large amounts of these materials will also reduce GC column life.

Separation due to polarity with silica gel (16) or Florisil (15) is used to separate the saturated hydrocarbons from the aromatic or polar priority organics. A cesium silicate approach (15), has also been employed to separate the acids (phenols) from the base/neutrals priority organics and from neutral interferences.

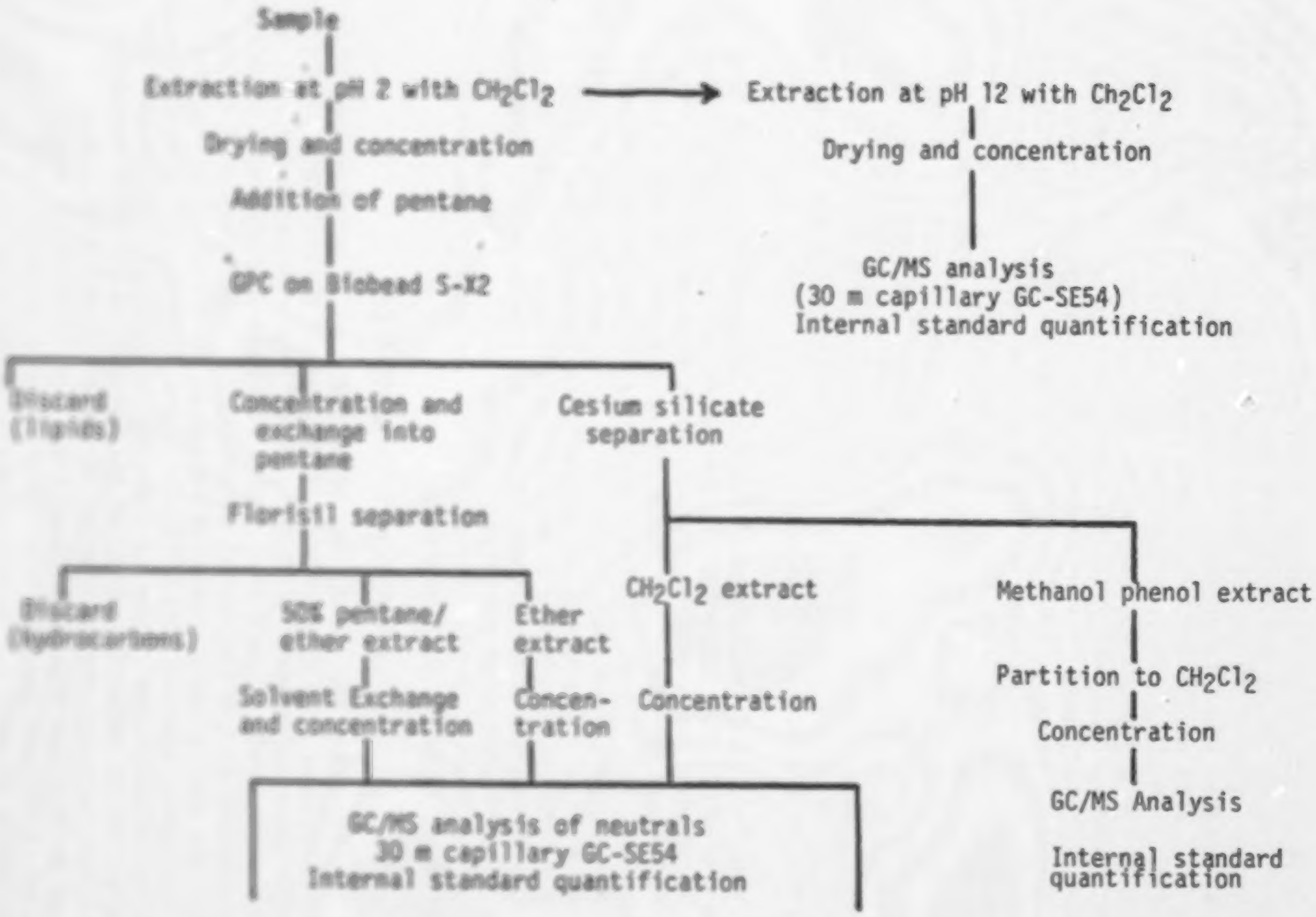
The separation or "clean-up" approaches can be assembled in various combinations to reduce the interferences from extracted municipal sludges. A method has been developed by DeWalle and Chian (15) for the analysis of the extractable priority organics in complex samples. The methodology Table 10, uses an acid/neutral extraction followed by a base extraction; gel permeation chromatography (GPC) of the acid/neutral extract into two fractions and a discard which contains the large interferences; florisil chromatography of one GPC fraction for separation of the saturated hydrocarbons from those priority neutrals in the fraction; and cesium silicate for separation of the acids (phenols) from the priority neutrals in the second GPC fraction. The phenol fraction from the silicate separation may be derivatized with diazomethane before GC/MS analysis or analyzed by fused silica capillary column GC without derivatization. The method produces three neutral fractions which may be combined into a single extract before GC/MS analysis or analyzed separately. The method uses capillary GC/MS techniques for the final analysis. The pesticides and PCB's are analyzed in the neutral fraction. Alternative extraction techniques under evaluation include homogenization-centrifugation, liquid/ liquid extraction and extractive steam distillation.

The Municipal Environmental Research Laboratory uses the DeWalle and Chian procedures for measuring the purgeable and extractable organics in both wastewater and sludge samples in the survey of 25 cities. The data indicate detection limits for the priority organics of about 1 μg/l for the extractables and <1 μg/l for the purgeables in wastewater samples. The data also suggest detection limits of 5-10 μg/l for the organics in sludge samples. The method is not fully satisfactory for all of the priority organics in all the highly variable sludge matrices. Losses of individual organics occur either through reaction with the matrix or losses in the separation processes. At the present time, insufficient data have been assembled to provide statistical analysis of the recoveries of the priority organics by the method.

OCCURRENCE IN WASTEWATER

Shortly after the establishment of the list of priority pollutants as the chemicals of most concern to EPA, it became obvious that a data base of occurrence and removal in municipal wastewater treatment plants was urgently required. Toward that goal three major surveys were undertaken; one conducted by the Cincinnati Laboratories, consisting of a survey of 25 cities and another more extensive survey of 40 cities is being carried by the Washington Headquarters Office of Water Planning and Standards and, finally, an additional study is underway in which six cities have been selected for an intensive study of the sources of priority pollutants. The study also includes the monitoring of the treatment plant. None of these survey studies has been completed, thus, the information available at this time is incomplete and firm conclusions cannot be made. Additionally, other survey studies for selected metals on the priority pollutant list are concurrently being conducted by intensive sampling at single plants

TABLE 10. ANALYSIS OF EXTRACTABLE ORGANICS WITH CLEAN-UP AND CAPILLARY GC/MS


Sample
Extraction at pH 2 with CH_2Cl_2
Extraction at pH 12 with Ch_2Cl_2
Drying and concentration
Drying and concentration
Addition of pentane
GC/MS analysis
(30 m capillary GC-SE54)
Internal standard quantification
GPC on Biobead S-X2
Discard (lipids)
Concentration and exchange into pentane
Cesium silicate separation
Florisil separation
Discard (Hydrocarbons)
50% pentane/ ether extract
Ether extract
CH_2Cl_2 extract
Methanol phenol extract
Solvent Exchange and concentration
Concen- tration
Concentration
Partition to CH_2Cl_2
Concentration
GC/MS Analysis
Internal standard quantification
GC/MS analysis of neutrals
30 m capillary GC-SE54
Internal standard quantification

in order to establish data on removals of metals by unit treatment processes. All literature, both published and that available in government, state and local records, is being searched for information on the occurrence and removal of priority pollutants. Thus far, much of this information is concerned with metals.

At the completion of all of these studies, and others, a fairly clear picture will emerge on the extent of the problem of priority pollutants in wastewater. These surveys will constitute the data base which will improve planning and environmental regulation.

Selection and Sampling of Survey Cities

Considering that there are some 20.6 thousand municipal treatment systems in the United States, it was important that the small number of cities that would be sampled (25 + 40 = 65 cities) would constitute a reasonably representative sampling of the country. Several criteria were established, including, size of flow, geographic location, proportion of industrial waste discharged to the plant including differing types of industrial discharges, type of treatment including primary, activated sludge, trickling filter, physical-chemical processes and degree of treatment quality being achieved. The final selection of the cities to be sampled is depicted in Fig. 2 showing the geographic locations and names of the cities. The plant locations of both the 40- and 25-city surveys are shown.

In the 25 city plant survey a single 24-hour composite sample is being taken from influent, primary effluent and final effluent. A grab sample is taken from the digester. Where chlorination of final effluent is practiced, a sample is taken both before and after chlorination. Fifteen of the cities will be resampled. The 40 city survey is designed similarly except each plant will be sampled for one week.

Data collection on the six city survey will concentrate on the wastewater collection system, as well as the municipal plant with the objective of determining the sources of the pollutants; i.e., the amounts and kinds of compounds contributed by industrial, commercial and domestic sources.

Since the samples were to be 24-hr composites, there was some concern about the losses from the sample of the more volatile components that are commonly present in wastewater; for example, chloroform or trichloroethylene which are known to have half-lifes in open vessels of only 20-30 minutes. To circumvent these anticipated losses a special sampler was developed at our laboratory (17). The sampler concept is simply a closed increasing-volume container obtained by using the principle of a piston schematically illustrated in Fig. 3 . A further modification was developed on the 25 city survey and incorporated into a novel sampler system. In operation, four or more samplers are located at various sampling points at the plant. Sampling times are radio-transmitted to each sampler and controlled by a mini-computer which determines the sampling sequence based on time of flow through the plant.

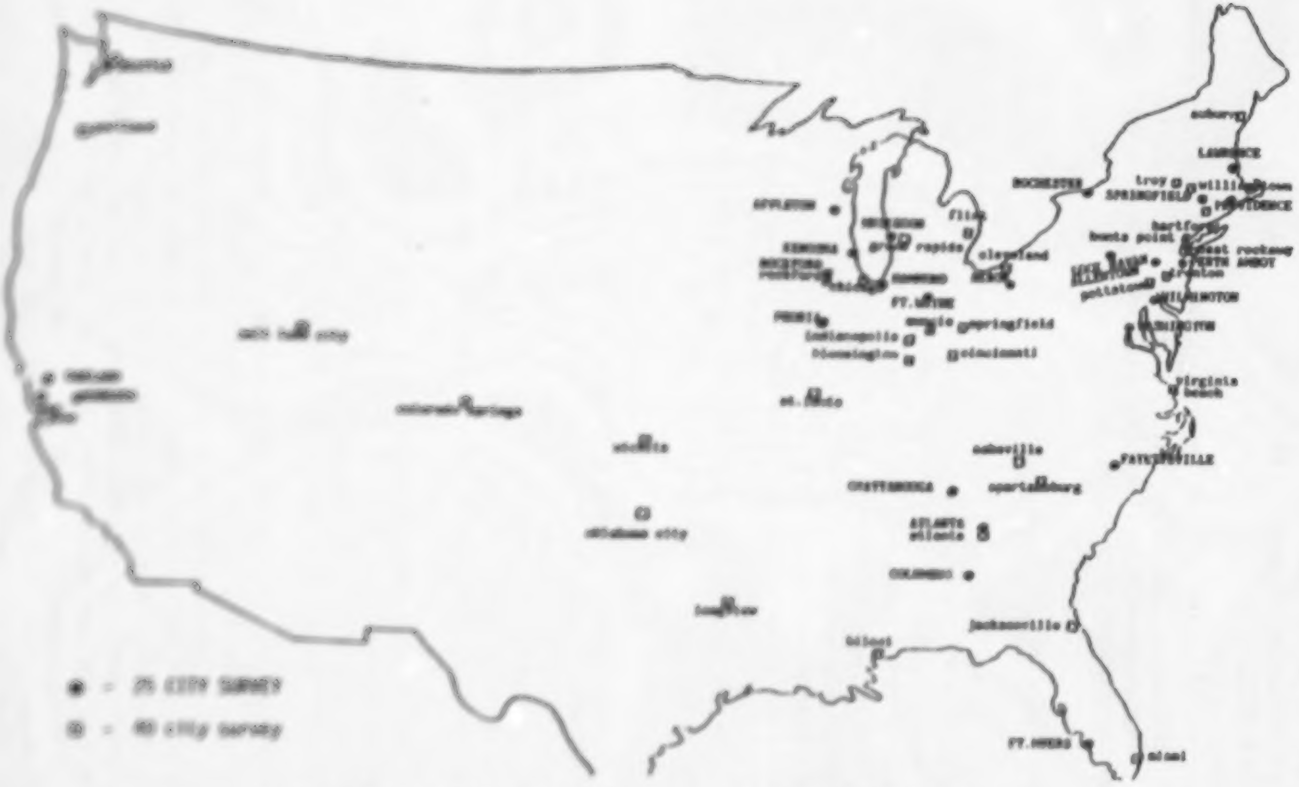
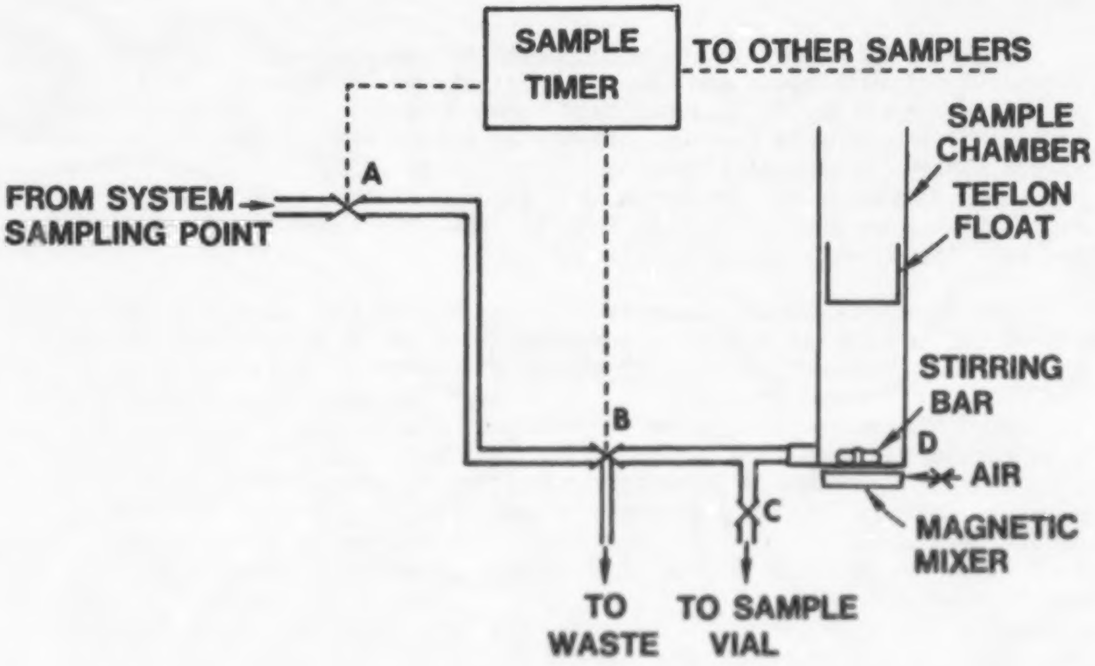

APPLETON
ROCHESTER
LAWRENCE
troy
SPRINGFIELD
PROVIDENCE
hartford
hunts point
east rockaway
PERTH AMBOY
trenton
pottstown
WILMINGTON
WASHINGTON
virginia beach
FAYETTEVILLE
flint
grand rapids
cleveland
KENOSHA
ROCKFORD
FT. WAYNE
PEORIA
muncie
springfield
indianapolis
bloomington
cincinnati
st. louis
asheville
CHATTANOOGA
spartanburg
ATLANTA
atlanta
COLUMBUS
jacksonville
biloxi
FT. MYERS
miami
salt lake city
colorado springs
wichita
oklahoma city
● = 25 CITY SURVEY
⊡ = 40 city survey

Figure 2. Locations of Cities Being Surveyed for Priority Pollutants


SAMPLE
TIMER
TO OTHER SAMPLERS
SAMPLE CHAMBER
TEFLON FLOAT
FROM SYSTEM SAMPLING POINT
A
STIRRING BAR
B
D
AIR
C
MAGNETIC MIXER
TO WASTE
TO SAMPLE VIAL

VALVE	TYPE	FUNCTION
A	2-WAY SOLENOID	DIRECT SAMPLE TO VALVE B
B	3-WAY SOLENOID	DIRECT SAMPLE TO CHAMBER
C	MANUAL TOGGLE	TRANSFER COMPOSITE SAMPLE TO VIAL & DRAIN REMAINDER
D	MANUAL NEEDLE	CONTROL MIXER SPEED

FIGURE 3. AUTOMATIC COMPOSITE SAMPLER FOR VOLATILE HALOGENATED ORGANICS

Survey Results

The 25 city survey is being conducted by the combined efforts of the University of Washington and Georgia Institute of Technology. A major portion of the study to date has been spent on developing the analytical procedures appropriate for the analyses of such difficult-to-analyze samples as raw wastewater and particularly sludges. The analytical implications have been discussed previously in this paper. At this time, twenty-five plants have been sampled but the results from only three are available, and even these data are incomplete.(15)

The Renton treatment plant in Seattle, Washington, serves a population of 198,000 and has an average dry weather flow of 136,000 m^3/day (36 mgd), 5% of which is contributed by industrial discharges. There are 37 permits for industrial waste discharge to the plant. The plant is a conventional activated sludge system with an aerated grit chamber and with effluent disinfection by chlorine and SO_2 dechlorination. Sludge is not treated at this plant but is pumped to another plant for treatment and disposal. The plant is well operated and produces a good effluent; average influent BOD of 273 mg/l is reduced to 12.5 mg/l and suspended solids from 351 mg/l to 12.5 mg/l. In addition to the conventional parameters, samples were analyzed only for the priority pollutants. Thus, other synthetic organic compounds may be and almost certainly were, also present - only the priority pollutants were identified and quantified. Since the tapes from the G.C-Mass Spectrometer are available as well as the individual solvent extracts, it is possible, at the conclusion of this phase of the study, to reanalyze the samples for compounds other than those on the priority pollutant list.

The Oakland, California, plant serves a population of 575,000 with a dry weather flow of 235,000 m^3/day (62 mgd) of which 15% is contributed by industrial discharges. The plant is an activated sludge system but varies from the previous plant in that the system includes prechlorination (for odor control but also providing the potential for forming chloro-organic compounds), aerated grit chamber (with the potential for loss of volatile organics), pure oxygen activated sludge aeration and final effluent chlorination and dechlorination. Influent BOD and suspended solids of 330 mg/l and 470 mg/l are reduced to 8 mg/l and 21 mg/l, respectively. Primary and secondary sludges are anaerobically digested, vacuum filtered and disposed to sanitary landfill. This plant differs, therefore, from the Renton plant in that the main flow contains sludge return flows.

The third plant sampled was the Clayton treatment plant in Atlanta, Georgia. The plant treats a flow from a population of 367,000 with a dry weather flow of 246,000 m^3/day (65 mgd). Percent industrial contribution is not available at this time but some indication is the 122 permits issued for industrial wastes discharge. The system consists of grit removal, primary sedimentation and pure oxygen activated sludge aeration. Chlorination is added for disinfection. Primary and secondary sludges are combined and anaerobically digested, dewatered by centrifugation and incinerated. The raw wastewater, in contrast to the previous two plants, is relatively weak with a BOD and suspended solids content of 122 mg/l and 141 mg/l which are reduced to 31 mg/l and 40 mg/l, respectively.

The data on all of the plants, for all of the priority pollutants and sampling points are summarized in Table 11. The sludge analyses varied with the plant. For the Seattle plant, untreated and combined primary and secondary sludge was sampled. For the Oakland and Atlanta plants the sample was taken after anaerobic digestion. Furthermore, the data for sludge are reported as micrograms per liter of liquid sludge. These data will subsequently be recalculated on the basis of dry solids, but this information is not now available. If one assumes that the solids concentration of the sludge was on the order of 3-4%, then the sludge values (on a dry weight basis, ug/kg) would be higher by a factor of 33-25.

The analyses, to date, covering the first three plants found 87 priority pollutants of the 127 compounds on the list (asbestos and dioxin were not analyzed). Not unexpectedly, there is a wide variation among plants for compounds identified and in concentrations found. Similarly, removals for specific compounds, varied from zero to almost 100%. Notwithstanding these variations some tentative observations can be made. Of the 87 compounds found, 24 organics and 13 metals were found in all of the plants. Chlorination for disinfection generally increased the concentration of certain compounds. While most compounds were detected at increased concentrations in the sludge, the phthalate esters and the polynuclear compounds tended to accumulate in the sludge to much greater concentrations. These trends will become more apparent and conclusive as the data from the remaining plants become available.

An attempt was made to identify those compounds in the plant effluents (after chlorination) which were equal to or exceeded the concentration for which criteria have been proposed for freshwater or marine aquatic life or for human health effects (Table 4). In essence, this list points out the compounds which may require some special attention. This information is presented in Table 12 in which the maximum concentration of the compound found in any of the effluents (after chlorination) from the plant survey is shown in the first column. Column two lists the aquatic (freshwater or saltwater) criterion which is equal to or exceeded by the effluent concentration. Generally, the 24-hour average value was selected. The ratio of effluent concentration to the aquatic life criterion, shown in column three, provides an indication of the amount of dilution required to reduce the effluent concentration to the criterion concentration. This calculation was repeated for human health effects criteria, where this applied for the particular compound, and are shown in columns four and five. Column six shows the range of percent removals obtained at the three treatment plants. Thus, a presumption can be made that whenever removal of a compound exceeds ~91% at the treatment plant and the effluent concentration still exceeds some criteria, then this compound may require special attention.

TABLE 11. OCCURENCE AND REMOVAL OF PRIORITY POLLUTANTS AT THREE TREATMENT PLANTS

No.	Compound	Plnt[a]	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge[b]	% Rem.[c]	Tox.
			Concentration, μg/l					
1	Acenaphthene	R	0.1					
		O	0.3		2.9			
		C	0.9	0.05	0.04	21.5	94	
4	Benzene	R	8.8	3.0	8.9	5.2	66	x
		O	10.5			12.4		
		C	7.7+	0.6	1.1		92++	
6	Tetrachloromethane	R				0.1		
		O	0.6					
		C						
7	Chlorobenzene	R						
		O	177.3	0.2	0.5	22.8	99.9	
		C						
8	1,2,4-Trichlorobenzene	R						
		O		0.3		13.5		
		C	0.6	0.1	0.3		85	
10	1,2-Dichloroethane	R	1.5	0.5	1.7		65	
		O	0.5	2.0	6.0		0	
		C	8.4+	1.3	1.1	+++	85++	
11	1,1,1-Trichloroethane	R	9.1	11.2	32.6	12.1	0	
		O	88.4	26.9	36.3		70	
		C	1791+	36.1	30.8	+++	98++	
13	1,1-Dichloroethane	R		0.4	1.8			
		O	1.7	1.6	1.9	2.3	6	
		C	6.7+	0.6		+++	92++	
14	1,1,2-Trichloroethane	R						
		O						
		C		2.5	1.8	+++		
16	Chloroethane	R						
		O	0.9					
		C						
18	Bis(2-chloroethyl)ether	R	0.1					
		O	0.12	0.01	0.01		92	
		C		0.1				

TABLE 11 (Contd.)

No.	Compound	Plnt[a]	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge[b]	% Rem.[c]	Tox.
			Concentration, µg/l					
21	2,4,6-Trichlorophenol	R						
		O	14.3					
		C						
23	Trichloromethane (chloroform)	R	4.2	4.0	7.5	4.5	5	x
		O	37.5	19.1	34.2	0.5	49	x
		C	17.1+	0.4	0.6	+++	98++	
25	1,2-Dichlorobenzene	R	4.8	3.0	3.9		38	
		O	1.8	1.3	0.2	15.8	28	
		C	43.9	1.2	10.1	312	97	x
26	1,3-Dichlorobenzene	R	0.7					
		O	0.2	0.4		5.5	0	
		C	3.1	0.4	1.3	17.0	86	x
29	1,1-Dichloroethylene	R		0.5	3.9			x
		O	1.8			1.1		
		C	43.2+	1.2	0.6	+++	97++	
30	1,2-Trans-dichloroethylene	R		0.3	0.2	1.6		
		O						
		C	0.9+			+++		
31	Dichlorophenol	R						
		O						
		C				9.4		
32	1,2-Dichloropropane	R						
		O		1.2				
		C						
34	2,4-Dimethylphenol	R						
		O	58.2					
		C						
36	2,6-Dinitrotoluene	R				42.6		
		O	2.6					
		C						
37	1,2-Diphenylhydrazine	R						
		O				0.02	0.01	
		C		0.1			3.8	

TABLE 11 (Contd.)

No.	Compound	Plnt[a]	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge[b]	% Rem.[c]	Tox.
			Concentration, µg/l					
38	Ethylbenzene	R	0.18	0.17	1.2	1.5	6	
		O	148.3	0.3	0.8	161	99.8	
		C			1.3			
39	Fluoranthene	R	0.2					
		O	3.1			42.3		
		C	0.6	0.05	0.1	27.2	17	x
42	Bis(2-chloroisopropyl) ether	R	0.16	0.06			62	
		O		0.3	o.2			
		C		0.3	0.1			
43	Bis(2-chloroethoxy) methane	R	0.31	0.02				
		O						
		C						
44	Methylene chloride (dichloromethane)	R	293	304	327		0	x
		O	9.0	4.6	7.3	0.2	49	x
		C	647*	182	141	***	72**	x
47	Bromoform	R						
		O		1.3		7.6		
		C						
48	Dichlorobromomethane	R						
		O	1.5		0.9			
		C	0.7*			***		
49	Trichlorofluoromethane	R	1.5					
		O	34					
		C	59*			***	80**	
50	Dichlorodifluoromethane	R						
		O			3.5			
		C	9.2*			***		
55	Naphthalene	R	1.3	0.2		11	85	
		O	6.8			76		
		C	10.9	0.1	0.4	319	62	
56	Nitrobenzene	R						
		O	0.3					
		C	0.1	0.01			90	

TABLE 11 (Contd.)

No.	Compound	Plnt[(a)]	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge[(b)]	% Rem.[(c)]	Tox.
			Concentration, µg/l					
57	2-Nitrophenol	R				1.4		
		O						
		C						
60	4,6-Dinitro-o-cresol (4,6-Dinitro-2-methylphenol)	R		5.3		7.7		
		O						
		C				0.1		
61	N-nitrosodimethylamine	R						
		O		0.4				*
		C						
62	N-nitrosodiphenylamine	R	0.2					
		O	1.3					
		C			0.2	123		
63	N-nitrosodi-n-propylamine	R	0.5	0.08	0.3	60	88	
		O	0.7	0.1			98	
		C			0.1			
64	Pentachlorophenol	R		12	15			*
		O						
		C	30		13			*
66	Bis(2-ethylhexyl) phthalate	R	1.3	0.8	13	40	31	
		O	270		3.4	7400		
		C	90	11		3800	70	
67	Butyl benzyl phthalate	R	6.7			20		
		O	120		0.4			
		C	18	1.6	3.7	21000	91	
68	Di-n-butyl phthalate	R	0.6	1.0	0.6	120	60	
		O	11	8	4.9	1.8	45	
		C	13	5	1.2	2000	62	
69	Di-n-octyl phthalate	R				120		
		O	11			290		
		C						
70	Diethyl phthalate	R	0.6	0.2		0.8	98	
		O	0.8		0.1			
		C	1.6	0.3	0.2		79	

TABLE 11 (Contd.)

No.	Compound	Plnt[(a)]	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge[(b)]	% Rem.[(c)]	Tox.
			Concentration, µg/l					
71	Dimethyl phthalate	R	0.7			4.2		
		O	0.4		0.1	0.8		
		C		0.1	0.1			
72	Benzo(a)anthracene (1,2-Benzoanthracene)	R	0.1					
		O						
		C						
73	Benzo(a)pyrene	R						
		O	0.1			4.8		
		C				8.4		
74	3,4-Benzofluoranthene	R	0.1					
		O						
		C				15		
75	Benzo(k)fluoranthene	R						
		O	0.4			8.0		
		C				14		
76	Chrysene	R	0.1					
		O	0.7			14		
		C				23		
77	Acenaphthylene	R	0.1					
		O						
		C	0.2	0.04		20	80	
78	Anthracene	R						
		O	0.05					
		C		0.07		9.6		
80	Fluorene	R	0.2					
		O	0.6			5.8		
		C	1.7	0.1		51	94	
81	Phenanthrene	R	0.4	0.4			0	
		O	1.2			27		
		C	3.2	0.3	0.3	100	91	
84	Pyrene	R	0.2					
		O	0.8			25		
		C		0.1	0.2	40		

TABLE 11 (Contd.)

No.	Compound	Plnt[a]	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge[b]	% Rem.	Tox.
			Concentration, µg/l					
85	Tetrachloroethylene	R	6.4	20	24	38	0	x
		O	49	10	25		80	x
		C	560+	5.3	4.4	+++	99++	x
86	Toluene (Methylbenzene)	R	0.8	8.4	11	36	0	
		O	112	1.0	1.0	126	99	
		C	70+	0.7	1.5	+++	99++	
90	Dieldrin	R	0.02			0.02		
		O	0.05		0.1			
		C		0.05	0.04	0.06		
91	γ-Chlordane	R	0.21	0.20	0.1		5	x
		O	0.5	0.2		0.7	50	
		C	0.3	0.2	0.8	0.7	33	x
91	α-Chlordane	R	0.2	0.1		0.2	50	
		O	0.4					
		C	0.1	0.3	0.6	0.5	0	x
91	Chlordene	R		0.1		0.5		
		O	1.1	0.1	0.1	0.1	91	x
		C		0.2	0.1			x
92	4,4'-DDT	R	0.5					
		O				0.6		
		C		0.4	0.5			
93	4,4'-DDE	R	0.2					
		O						
		C		0.05	0.2			
94	4,4'-DDD (p,p'-TDE)	R	0.2					
		O	0.3	0.2		0.3	87	
		C	0.17	0.14	0.2	0.2	17	
95	a-Endosulfan-Alpha	R	0.1	0.01		0.04	91	
		O	0.28	0.04		0.4	86	
		C		0.01		0.2		
96	b-Endosulfan-Beta	R						
		O	0.23	0.14		0.4	39	
		C		0.1	0.04	0.05		x

TABLE 11 (Contd.)

No.	Compound	Plnt$^{(a)}$	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge$^{(b)}$	% Rem.$^{(c)}$	Tox.
			Concentration, µg/l					
97	Endosulfan sulfate	R						
		O				1.3		
		C			4.8			x
98	Endrin	R	0.2		0.2			x
		O						
		C			0.4			x
100	Heptachlor	R	37		0.01			x
		O						
		C						
101	Heptachlor epoxide	R	0.15	0.08	0.08		47	
		O	5	0.40			92	
		C			0.36	0.26		
102	a-BHC-Alpha	R	0.13			0.16		
		O						
		C		0.13				
103	b-BHC-Beta	R	0.58					
		O						
		C			0.44			
104	γ-BHC-Gamma (lindane)	R	0.23			0.32		
		O	0.41	0.16	0.14	0.12	61	
		C	0.14	0.15	0.16	0.13	0	
105	g-BHC-Delta Δ-BHC	R		0.9	0.9			
		O			0.87			
		C						
114	Antimony	R	4	4.5	4	40	0	
		O	3.8	2.5	<1	120	34	
		C	<1	<1	<1	<1		
115	Arsenic	R	12	4.4	4.5	400	63	x
		O	14	11.3	11.3	1400	19	x
		C	60	30	30	5000	50	x
117	Beryllium	R	0.2	0.2	0.1	3.4	0	x
		O	<1	<1	<1	14	x	
		C	4	5.1	2.9	23	0	x

TABLE 11 (Contd.)

No.	Compound	Plnt$^{(a)}$	Infl. Sew.	Sec. Effl.	Sec. Effl. Chlor.	Sludge$^{(b)}$	% Rem.$^{(c)}$	Tox.
			Concentration, µg/l					
118	Cadmium	R	22	23	14	400	0	x
		O	34	<1	< 1	3000	< 97	
		C	1.1	<1	< 1	770		
119	Chromium	R	170	35	30	2200	79	
		O	390	41	66	1900	89	
		C	56	16	15	23,000	71	
120	Copper	R	100	1	1	2100	99	
		O	140	140	70	15,000	0	x
		C	21	31	31	2500	0	x
121	Cyanide	R	29	8	0	120	72	
		O	126	65	77		48	x
		C	28	7	4	20,000	75	x
122	Lead	R	12	14	6	190	0	
		O	96	13	25	4000	86	
		C	117	47	22	13,000	60	
123	Mercury	R	11	2	1	85	82	
		O	17	13	10	270	24	
		C	9.6	0.7	1.6	480	93	
124	Nickel	R	105	80	70	9500	24	
		O	640	290	150	35,000	55	x
		C	560	1400	865	99,000	0	x
125	Selenium	R	1	1	1.8	9	0	
		O	<1	<1	<1	85		
		C	3.8	3.8	7	<1	0	
126	Silver	R	17.3	1	1	213	94	x
		O	9.3	2.1	<1	250	77	
		C	9.6	7.1	7.1	400	26	x
127	Thallium	R						
		O	<1	<1	<1	<1		
		C	<1	<1	<1	<1		
128	Zinc	R	125	50	25	3100	60	
		O	1600	150	200	120,000	91	
		C	1700	500	300	340,000	71	

1. (a) R=Renton, WA; O=Oakland, CA; C=Clayton, GA.
2. (b) Sludge reported on liquid basis in µg/l
3. (c) % Removal calculated from Infl.Sew. to Sec. Effl.
4. + Conc. in Primary Effluent; Sewage data lost.
5. ++ % Removal from primary effluent.
6. +++ Data lost.
7. Blank spaces indicate compound not detected
8. Column labeled "Tox." indicates that concentration in the effluent was approximately equal to or exceeded the limits for fresh or saltwater aquatic life or limits for human health effects.

TABLE 12. COMPOUNDS FOUND IN EFFLUENTS THAT EXCEED SOME ENVIRONMENTAL CRITERION

Compound	1 Max Conc In Effluent μg/l	2 Aquatic Criteria μg/l	3 Ratio(a)	4 Human Health Effects μg/l	5 Ratio(a)	6 Range(b) % Removal
Benzene	8.9			15	0.6	66-92
Chloroform	34			2.1	16	5-98
1,2-Dichlorobenzene	10	15	2.3			
1,1-Dichloroethylene	3.9			1.3	3	97
Fluoranthene	0.1	0.3	3			17
Methylene chloride	327(c)			2.0	16	49-72
N-Nitrosodimethylamine	0.4			0.026	15	-
Tetrachloroethylene	25			2.0	13	0-99
Chlordane (tech)	0.8	0.009	89	0.001	800	5-50
Endrin	0.4	0.002	200			-
Heptachlor	0.01	0.0015	7	0.00023	44	-
Arsenic	30	29	1	0.02	1500	19-63
Beryllium	2.9			0.087	33	-
Cadmium	14			10	1.4	>97
Copper	70	0.79	89			0-99
Cyanide	77	1.4	55			48-75
Nickel	865			133	6.5	0-55
Selenium	7	4.4	1.6			-
Silver	7.1	0.009	789	10	0.7	26-94

TABLE 12 (Contd.)

Compound	1 Max Conc In Effluent µg/1	2 Aquatic Criteria µg/1	3 Ratio(a)	4 Human Health Effects µg/1	5 Ratio(a)	6 Range(b) % Removal
Chromium	66(Total)			50	0.8	71-89
Acenaphthene	2.9			20×10^{-3}	145	94
1,2-dichloroethane	6.0			7	0.9	0-85
1,1,2-trichloroethane	1.8			2.7	0.7	-
naphthalene	0.4			143×10^{-3}	2.8	62-85
Dieldrin	0.1			4.4×10^{-8}	230×10^{4}	-
4,4-DDT	0.5			0.98×10^{-3}	510	91
α-BHC-alpha	0.13			16×10^{-3}	8	-
β-BHC-beta	0.44			28×10^{-3}	16	-
γ-BHC-gamma	0.16			54×10^{-3}	3	0-61
δ-BHC-delta	0.9			21×10^{-3}	43	-

(a) Ratio of effluent concentration to aquatic or health criterion
(b) Range of percent removal in three city treatment plants
(c) High values may be due to contamination from analytical extraction solvent

REMOVABILITY - TREATABILITY

The municipal wastewater treatment plant physically occupies a central position in the scheme to control hazardous and toxic substances in liquid wastes. Industrial discharges (even after pretreatment) and commercial and household wastes generally enter a municipal treatment plant for treatment. How well the receiving stream (and ambient air surrounding the treatment plant) is protected against contamination by synthetic organic and metal compounds depends on the effectiveness of wastewater treatment processes for removal of these substances. There is, therefore, the unavoidable need to assess the effectiveness of conventional and unconventional wastewater processes to reduce influent concentrations to levels acceptable for discharge to surface waters.

While the terms "removability" and "treatability" have commonly been used interchangeably, there is a justifiable reason (for research purposes) to make a distinction between the two terms. It is useful to consider that treatability is a property or characteristic of the molecule, in much the same way as solubility or vapor pressure. Treatability of a compound is determined by specified procedures under controlled conditions and thus compounds can be compared or ranked relative to other compounds. The position in a ranking of compounds can provide some insights on the behavior in a certain treatment process. For example, a compound which adsorbs on carbon by batch isotherm testing, at 300 mg/g of carbon is more likely to be removed from solution than a compound that is adsorbed at 10 mg/g. Such treatability research is necessary in order to provide the data base to formulate more vigorous modeling of how compounds in general behave in a treatment process.

It is recognized that the treatability of a compound, much like its solubility, will change when conditions change; such as mixture with other compounds, aqueous composition, pH, etc.

Removability is a more general term which simply describes the change (or no change) in concentration of a compound between entering a process and leaving a process. Several treatability mechanisms may, and usually do, operate in a process to achieve a change in concentration. A good example is the activated sludge process. A change in concentration of a compound between influent and effluent can occur via several treatment mechanisms including: stripping, biodegradation, adsorption, precipitation, hydrolysis, etc. Thus, removability will be highly characteristic of the conditions of the process at some time and place.

Research on the 114 organic compounds on the priority pollutant list was planned to occur in phases. The first phase consists of a series of "screening" tests for all of the compounds for all of the three processes being considered (biodegradation, carbon adsorption, air stripping). Most of this phase has been completed. The preliminary results will be discussed in this paper. The second phase will consist of dynamic rather than static or batch tests and will include more complex solutions of mixtures of compounds and wastewaters rather than distilled water test solutions. This phase has not yet been started. The third phase will consist of "removability" tests in which treatment systems rather than individual unit processes will be evaluated for removal of mixtures of organic compounds in wastewater. Some preliminary research has been completed in this phase and additional work is being conducted at pilot plant scale at our new Test and Evaluation facility located in Cincinnati, Ohio.

Biodegradability

Perhaps the single most important process for control of toxic organic compounds is biological decomposition. Thus, the biodegradability of a compound is an essential element of information. It is also important, in the test for biodegradability, to isolate insofar as it is possible, other mechanisms of removal that occur in operating biological plants, such as adsorption on sludge and stripping from solution. The screening test selected to meet these criteria, is the simple static flask procedure in which the compound is dissolved (or emulsified) into BOD dilution water (described in Standard Methods) supplemented with 5 mg/l of yeast extract. This medium supplies sufficient nutrients for active biological growth but will not produce excess solids.

The test compound (usually at concentrations of 5 and 10 mg/l) and nutrient solution contained in 250 ml flasks, either cotton stoppered or glass stoppered for the volatile compounds, were inoculated with 10 ml of settled domestic sewage. The flasks were incubated in the dark in a constant temperature room at 25^{o}C for seven days. The flasks were shaken daily to ensure homogeneous distribution of the culture. At the end of the first incubation period, part of the contents of the flask was used to analyze for residual test compound, and another portion was used as inoculum for a fresh flask. Three weekly subcultures were prepared resulting in a total elapsed time of 28 days for each test. The usual analytical procedure consisted of solvent extraction, solvent volume reduction and completion of the analysis by gas chromatography.

At this time, the experimental work on the screening test for biodegradability is nearing completion, and is continuing, but preliminary data are available for most of the priority pollutant compounds. (18) Some typical results are shown in Table 13 where the data for a single class of compounds, the halogenated ethers, are shown. Bis(chloromethyl)ether, the remaining ether compound in the priority pollutant list, is the most active carcinogenic ether, and will be determined at a later date under special safety precautions.

Several types of reactions to biodegradation can be deduced from the screening test. Both bis-(2-chloroethyl)ether and bis-(2-chloroisopropyl) ether are readily degradable and 2-(chloroethyl vinyl)ether is degradable after a short acclimation period. The remaining three ether compounds are not degradable with bis(2-chloroethoxy)methane being completely resistant to degradation within the 28-day acclimation period.

TABLE 13. BIODEGRADABILITY OF HALOGENATED ETHERS BY STATIC FLASK TEST

		Percent Degradation of Compound in 7 Days			
Test Compound[a]	Performance	Original Culture	First Sub-Culture	Second Sub-Culture	Third Sub-Culture
Bis-(2-Chloroethyl) ether	D	100	100	100	100
2-Chloroethyl vinyl ether	D	76	75	100	100
4-Chlorodiphenyl ether	N	0	32	36	1
4-Bromodiphenyl ether	N	0	19	36	0
Bix-(2-Chloroethoxy) methane	N	0	0	0	0
Bis-(2-Chloroisopropyl) ether	D	85	100	100	100

(a) Test concentration - 5 mg/l

D Degradable
N Not significantly degradable

Some 90 compounds of the 114 organic compounds on the priority pollutant list have been completed or are in various stages of completion. Of these, 23 compounds, or 29% of the total tested, listed in Table 14, have been found to be either non-degradable within the 28 days of three successive subcultures or have shown less than 50% reduction in concentration indicating that microbial adaptation to the compound was slow. It should be clearly understood that the screening test has limitations, and it would be dangerous to extrapolate these data to continuous flow biological systems which contain high concentrations of active biological solids and where longer times are available for microbial acclimation to the compound. Nevertheless, it can generally be assumed that those compounds that are readily degraded in the screening test are likely to be degraded in continuous flow biological systems. The screening test also serves the purpose of identifying those compounds which, while they were not degradable in this test, will require the second level of testing by operating bench-scale continuous flow activated sludge systems. This approach is planned for the compounds that show little or no degradation in the screening test. Also shown on Table 14, column 2, are the carbon adsorption capacities of these compounds calculated at an initial concentration of 100 ug/l. The adsorption process is discussed in the next section.

Activated Carbon Adsorption

One process that is increasingly being considered to control synthetic organic compounds, especially for those compounds which are refractory to biological degradation, is activated carbon adsorption. Unfortunately, carbon is selective in its ability to adsorb compounds. The capacity, in terms of milligrams of compound adsorbed per gram of carbon, varies over 2 to 3 orders of magnitude and certain classes of compounds are not adsorbed at all. As another tool to evaluate the treatability of a compound it is necessary to determine its adsorption on activated carbon.

TABLE 14. NON-DEGRADABLE COMPOUNDS AND CARBON ADSORPTION CAPACITIES

Compound	Percent reduction after 3rd sub-culture	Carbon adsorption capacity[b] (mg/g)
4,6-Dinitro-o-cresol	52	76
1,2-Dichlorobenzene	35	47
1,4-Dichlorobenzene	16	41
1,2,4-Trichlorobenzene	24	78
Hexachlorobenzene	5	450
1,2-Benzanthracene	0	
Chrysene	59	
4-Chlorodiphenyl ether	1.0	61
4-Bromodiphenyl ether	0	30
Bis(2-chloroethoxy)methane	0	2.6
Polychlorinated biphenyls - PCB		
1016	48	
1221		48
1232		120
1242	66	
1248	0	
1254	0	
1260	0	
N-nitroso-di-N-propylamine	28[a]	16
Bromoform	59[a]	5.7
Dichlorobromomethane	55[a]	1.9
1,1,2-Trichloroethane	38[a]	1.4
1,1,1,2-Tetrachloroethane	27[a]	4.5
1,2-Dichloroethane	41[a]	0.5

(a) Data only for the results from first sub-culture = 14 days

(b) Adsorption capacity for 100 μg/l initial concentration at neutral pH

As of this date, adsorption isotherms have been obtained in almost all of the priority pollutants. (19) A report on this work is in preparation. This report will contain the activated carbon capacities at several initial concentrations of compound, the constant, K, and slope 1/n, of the powdered or granular carbon to reduce differing initial concentrations of compound to specified residual concentrations.

The most commonly used technique to determine adsorbability of compounds on carbon, or any other sorbent, is to determine an adsorption isotherm, which measures the extent to which an organic chemical partitions itself between the solid and solution phases of an aqueous solution of the compound. Experimentally, the test is conducted by contacting various amounts of carbon with a solution of the compound with a known initial concentration. After a period of contact, during which time the suspension is stirred, the carbon is separated from the suspension and the clear solution is analyzed for residual compound.

In our laboratory, a granular carbon from a single major manufacturer was pulverized until all of it passed a 200 mesh screen. Special care was taken to assure that the initial concentration of compound did not exceed its solubility. Solutions were prepared in distilled water in most cases. Special precautions were also taken for compounds that are volatile by using containers that were completely filled by the solution and were tightly capped during the contact period. For those compounds that are ionizable, the isotherm was determined at three pH's; 3.0, 7.0 and 9.0 by adjusting the solution pH with acid or base. After two hours of contacting time at room temperature, the carbon was separated by filtration through various types of filters. Residual compound was determined by a variety of analytical methods including TOC, U-V spectroscopy and gas chromatography by either the purge and trap technique or by solvent extraction, solvent volume reduction and G.C. detection.

The adsorption data were plotted according to the Freundlich equation. While this equation is empirical it is nonetheless widely used and has been found to describe adequately the adsorption process in dilute solution. The Freundlich equation has the form:

$$\frac{X}{M} = KC_f^{1/n}$$

The data were plotted according to the logarithmic form of the equation which has the form:

$$\log \frac{X}{M} = \log K + 1/n \log C_f$$

where:

$X = (C_o - C_f)\ V$ = initial concentration of solution in mg/l minus final concentration in solution at equilibrium multiplied by solution volume, V, in liters.

M = weight in grams of adsorbent (carbon).

C_f = final concentration of solute in mg/l at equilibrium

K = intercept at C_f = 1 (log C_f = 0)

1/n = slope of the line

For the dilute solutions in this study, this equation yields a straight line with a slope of 1/n and the intercept equal to the value of K when $C_f = 1$ ($\log C_f = 0$). The intercept K, is an indicator of adsorption capacity, expressed as milligrams of compound per gram of carbon. The slope, 1/n, indicates the intensity of adsorption. Linear least squares regression analysis was used to locate the line which also permitted the calculation of the correlation coefficient, r.

Whether a compound adsorbs on carbon and at what capacity depends on a multitude of parameters of both the carbon particle as well as the characteristics of the organic molecule. Much research remains to be done to understand the mechanisms which determine the adsorption characteristics of a compound. Active research, in this laboratory and others, is providing some answers. It is entirely likely that with sufficient basic information it will be possible to predict the adsorption behavior of any untested compound based simply on a knowledge of molecular characteristics. This line of research is being pursued in this laboratory with the ultimate objective of being able to predict the carbon adsorption behavior of the vast numbers of untested compounds and the continuing outpouring of newly developed organic compounds. Actual testing of these thousands of compounds is impractical, if not impossible, and a capability to predict is essential.

An example of the influence of pH and differing substitutions on the phenol molecule is shown in Table 15. The carbon capacity, X/M,is shown for phenols and substituted phenols at three pH's at an initial concentration of 10 mg/l for each compound. Two observations can be made. For an acidic ionizable compound such as phenol, adsorption capacity is maximum at acidic pH's and this behavior holds true regardless of the substitutions made to the original phenol molecule. A second observation is that, at neutral pH, the adsorption of phenol increases from 80 mg/g to 160 mg/g when two nitro groups are added to the molecule. When a 9-carbon alkyl group is added (nonyl), the capacity jumps to 595 mg/g. Addition of five chlorine atoms, as in pentachlorophenol, confers high adsorbability.

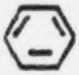
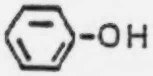
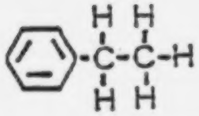
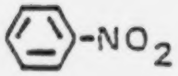
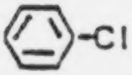
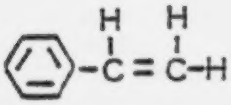
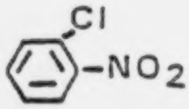
A further example of the effect of substitution on an aromatic compound is shown in Table 16. The carbon adsorption capacities, in mg/g, are shown for the compounds all at an initial concentration of 1 mg/l. The data show an increase in adsorption capacity as differing functional groups are added to a benzene parent molecule. It is clear that the changes in adsorption cannot be attributed to some simple explanation, such as molecular weight, since ethyl benzene and styrene side chains differ only slightly in molecular weight and in the presence of a C = C in the styrene substitution. The data suggest that, in addition to changes in atoms and simple groupings such as nitro- or hydroxo-, the adsorbability is also affected by molecular structural differences.

TABLE 15. ADSORPTION OF PHENOL AND SUBSTITUTED PHENOLS

Compound	pH	X/M, mg/g[(a)]	K	1/n	r
Phenol	3	85	12	0.86	0.92
	7	80	13	0.77	0.91
	9	70	22	0.49	0.94
2,4-Dinitrophenol	3	405	168	0.38	0.99
	7	160	18	0.95	0.94
	9	75	41	0.25	0.87
Nonylphenol	3	570	55	1.03	0.97
	7	595	254	0.37	0.98
	9	275	148	0.27	0.98
Pentachlorophenol	3	635	260	0.40	0.98
	7	385	145	0.42	0.98
	9	260	100	0.41	0.98

(a) Measured at 10 mg/l initial concentration.

TABLE 16. ADSORPTION CAPACITIES FOR BENZENE AND SUBSTITUTED BENZENES

Compound	Structure	Adsorption Capacity*(mg/g)
Benzene		1.0
Phenol		21
Ethylbenzene		53
Nitrobenzene		68
Chlorobenzene		93
Styrene		120
1-Chloro-2-nitrobenzene		130

*Measured at 1 mg/l initial concentration.

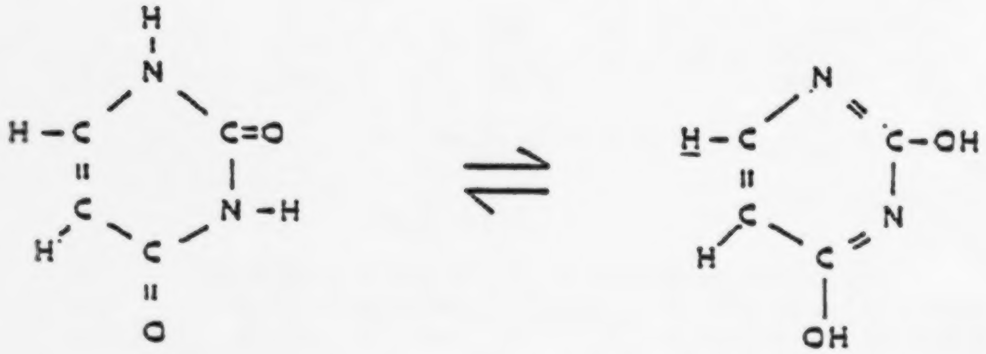
A final example is shown in Table 17 where adsorption capacities are shown for uracil and substituted uracils. While not a priority pollutant, the compound is a common component in sewage. Two observations can be made from the data. One, that adsorption capacity changes as the parent compound is halogenated and, further, that the kind of halogen atom - chlorine, bromine or fluorine - exerts its own characteristic influence on adsorption. Another point that should be made, although not apparent from the data shown, is that uracil is an innocuous metabolic byproduct in urine, but when halogenated, as does occur during disinfection, the compound becomes increasingly toxic. Fluorouracil is considered to be a mutagenic compound.

These examples demonstrate that adsorption capacity of a parent molecule changes as differing atoms are added and as differing structural changes are made to side chains. The problem is that, at this point, there is no immediately apparent reason why these changes occur.

In the previous examples, the compounds were tested individually where the compound adsorbed on the carbon free of any extraneous competition or interference. Realistically, compounds rarely occur alone in solution and competition with other compounds will almost always occur. The effects of these interactions are illustrated by Table 18, where adsorption capacity data are shown for six closely related halogenated methanes and ethane. These compounds are common artifacts of disinfection of water with chlorine. When the compounds were determined individually in a mineralized distilled water, the compounds can be arranged in order of increasing adsorption capacities. In this example, capacities are expressed as micrograms per gram of carbon at an initial concentration of 100 micrograms per liter. There is an order of magnitude difference between the least and most adsorbable compounds. When these same compounds are now dissolved together in a volume of mineralized distilled water at the same initial concentration of 100 ug/l for each compound and the adsorption for each individual is determined, the order of adsorption remains unchanged. Put another way, each compound in the mixture behaves as an individual compound. The only effect observed is that the capacities are reduced by approximately one-half. This indicates that there is competition among the compounds for adsorption sites, but since the ranking is preserved, each compound exerts its own degree of adsorption. And, finally, the last column shows the results of the mixture of compounds in clarified secondary treated wastewater. With the competition from the organics in the wastewater, the adsorption of the individual compounds is further reduced. But - with one exception - the order of adsorption capacity is preserved. The single exception may be due to the analytical difficulties of determining parts-per-billion of highly volatile compounds.

Continuing research in this area will provide the data base from which general conclusions can be made, thus providing the capability to predict the adsorption characteristics of any unknown compound. For prediction, the technique of structure-activity analysis is a promising approach.

TABLE 17. ADSORPTION OF URACIL AND SUBSTITUTED URACILS



Compound	X/M,mg/g	K	1/n	r
Uracil	55	9.3	0.76	0.84
5-Fluorouracil	65	2.0	1.52	0.88
5-Methyluracil	90	27	0.51	0.91
5-Chlorouracil	100	25	0.60	0.97
5-Bromouracil	130	49	0.43	0.98

C_0 = 10 mg/l pH = 7.0

X/M Decreased at higher pH values.

TABLE 18. ADSORPTION DATA FOR HALOHYDROCARBONS

Compound	Miner. Water Alone	Miner. Water Mixture	Clarif. Sec. Effluent Mixture
Dichloroethane	1070[a]	440	520
Chloroform	1580	930	365
Bromodichloromethane	3370	2560	875
Carbon tetrachloride	4660	-	-
Dibromochloromethane	7520	4540	885
Tribromomethane	28700	10800	1530

(a) $\frac{x}{m}$ values in μg/g at C_0 = 100 μg/l

VOLATILITY

Even a casual inspection of the organics on the list of priority pollutants will show that many of the low molecular weight chlorinated hydrocarbons are volatile compounds. It follows from this that loss of these compounds from solution can be an important route of removal from the aqueous flow, but this also indicates that the potential exists for contamination of the air around a wastewater treatment plant. Treatment plants offer ample opportunity for volatilization from processes such as the flows in the collection system, aerated grit chambers, settlers and most especially from aeration basins where the driving force for desorption from solution is provided by the air or oxygen aeration. Thus, it is not surprising that volatilization from water bodies to the atmosphere is generally recognized as a significant pathway for transfer of organics from one environmental medium to another.

The volatilization process from an aqueous solution is generally accepted as consisting of diffusion of the solute from the bulk of the water to the interface, followed by transfer across the interface and finally diffusion from the interface to the bulk of the air phase.

There have been many attempts in the literature to develop mathematical models which would predict the rate of volatilization of a compound from aqueous solution. Most models incorporate such parameters as Henry's Law constant, gas and liquid phase mass transfer coefficients and, more recently, models have incorporated coefficients which account for other parameters that affect the volatilization such as adsorption on solids and rate of biodegradation.

An important coefficient in virtually all of the models is the Henry's Law constant. This constant is an expression of the distribution of a volatile solute at equilibrium between the liquid and the vapor phases. For compounds with low solubility, Henry's Law constant can be calculated from the aqueous solubility of the compound and the vapor pressure of the pure compound, both at the temperature of the system. Unfortunately for many of the compounds of environmental interest, Henry's Law constant is not available, and furthermore, the information needed for calculating the constant - solubility and vapor pressure - are either nonexistent or erroneous.

The laboratory of the Wastewater Research Division in Cincinnati, Ohio, undertook the task of experimentally determining Henry's Law constant. (20) The procedure chosen for this work was described by Mackay, et al. (21) In brief, the procedure consists of stripping with nitrogen at a controlled rate of flow a solution of the test compound in a water-jacketed column maintained at a constant temperature of 25°C. The solution was sampled at timed intervals and the decrease of concentration due to the stripping was recorded. Various analytical methods were used depending on the compound being tested and included such procedures as U-V spectroscopy, purge and trap gas chromatography and solvent extraction, solvent volume reduction and gas chromatography using electron capture detection.

The test apparatus is shown in Fig. 4. Cylinder nitrogen flows through a flow control ratameter and then through a gas absorption bottle to saturate the gas with water, and is then delivered to the test solution through a glass frit which produces fine bubbles of gas. The solution is temperature controlled by circulating constant temperature water through a jacket around the solution. The sampling tube inlet is located approximately at the mid-depth of the solution. The solution was generally sampled and analyzed at timed intervals until the remaining concentration approached the limits of analytical detection. The curve is then determined by least squares regression analysis.

An example of the experimental results is shown in Fig. 5 for ethylbenzene where concentration shown as ln absorbance at 260 nm is plotted vs. time in minutes. Henry's Law constant was calculated by the following equation:

$$H = -\frac{\text{slope} \times VRT}{G} =$$

H = Henry's Law constant, $m^3 atm\ mol^{-1}$

slope = Change in solution concentration/minute

V = Volume of test solution, $m^3 = 9.925 \times 10^{-4}\ m^3$

R = Gas constant, $\frac{m^3\ atm}{mol\ ^oK} = 8.2 \times 10^{-5}\ \frac{m^3\ atm}{mol\ ^oK}$

T = Temperature in degrees K = 298 °K

G = Gas flow in $m^3/min = 1.0 \times 10^{-4}\ m^3/min$

Henry's Law constant for ethylbenzene calculated from the experimental data is

$6.40 \times 10^{-3}\ m^3\ atm\ mol^{-1}$

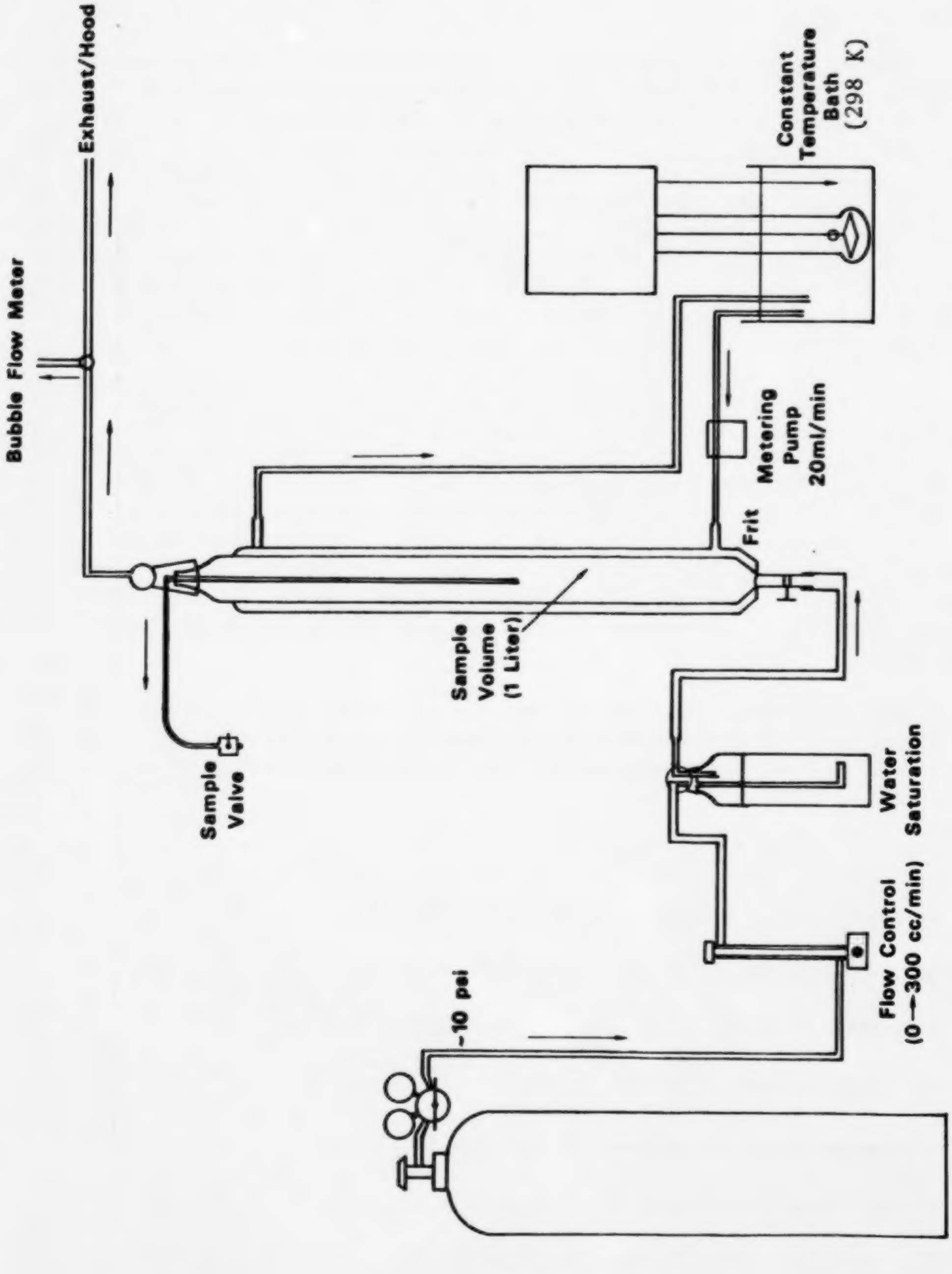

~10 psi
Flow Control (0—300 cc/min)
Water Saturation
Sample Valve
Sample Volume (1 Liter)
Frit
Metering Pump 20ml/min
Bubble Flow Meter
Exhaust/Hood
Constant Temperature Bath (298 K)

Figure 4. Apparatus For Experimental Determination of Henry's Law Constant

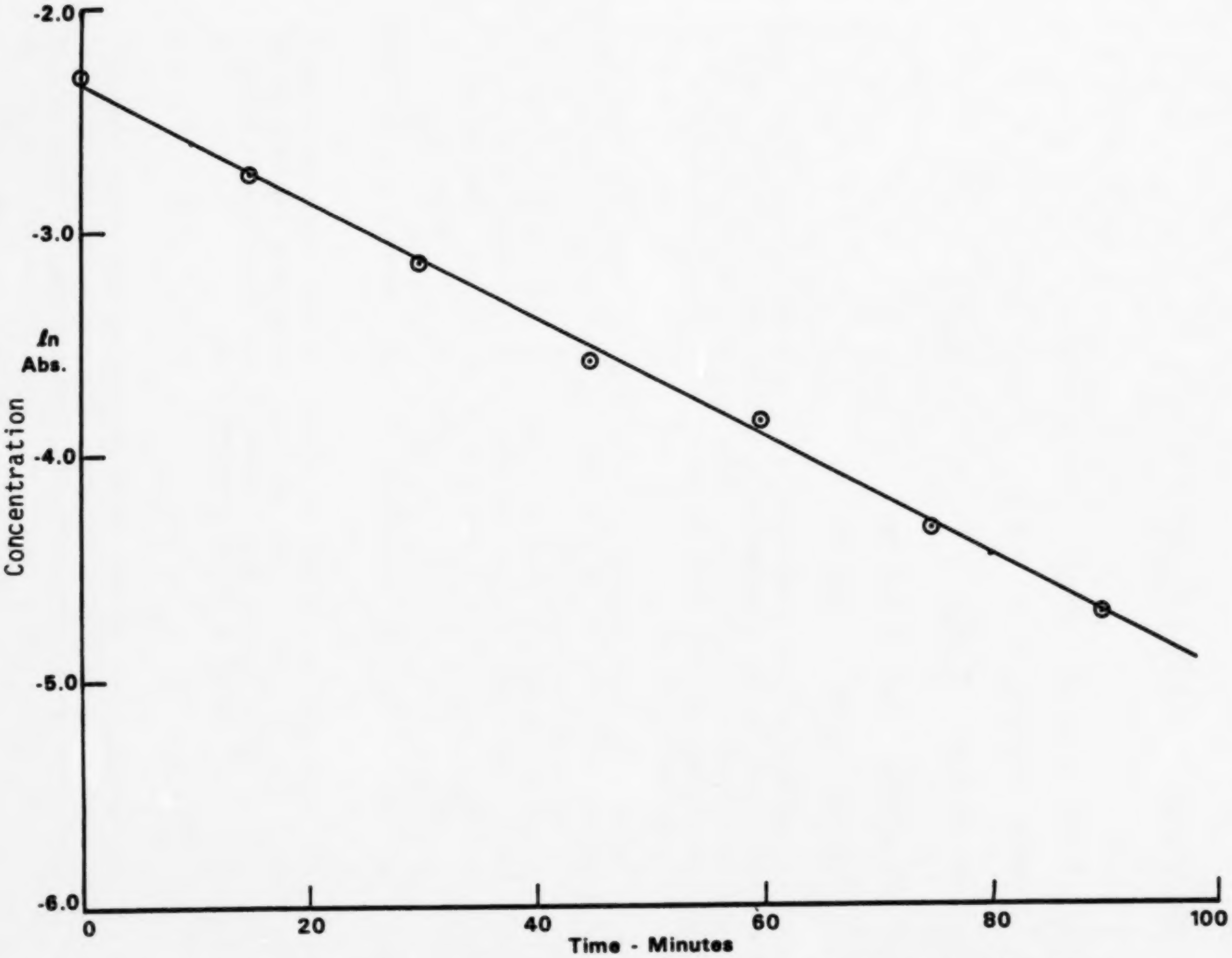


Figure 5. Determination of Henry's Law Constant For Ethylbenzene

When the necessary information on aqueous solubility and vapor pressure are available, Henry's Law constant can be calculated by the following equation:

$$\text{H (calculated)} = \frac{16.04 \times MW \times VP \times R \times T}{S \times T} =$$

where:

MW = Molecular weight, g/mol = 106.16 g/mol

VP = Vapor pressure, mm = 9.5 mm

R = Gas constant, $\frac{m^3 \text{ atm}}{\text{mol } ^oK}$ = 8.2×10^{-5} $\frac{m^3 \text{ atm}}{\text{mol } ^oK}$

S = Solubility, mg/l = 206 mg/l

T = Temperature, ^oK = 298 ^oK

In this example the calculated value is 6.44×10^{-3} m^3 atm mol^{-1} which corresponds very well to the experimentally determined value. Thus, whenever accurate values for solubility and vapor pressure are available the calculated Henry's Law constant will be essentially equivalent to the experimentally determined value.

The results of the experimental work are shown in Table 19. The compounds represent essentially all of the compounds on the priority pollutants list that have appreciable potential for volatility. An interesting observation is that certain compounds with very low vapor pressures, as for example the pesticides, have substantial volatility from aqueous solution because of the concurrent extremely low aqueous solubility. The column labeled "calculated" in the Table 19 illustrates the difficulty and inaccuracy in calculating the constant. The basic information is either lacking, inaccurate or involves the problem of vapor pressure extrapolation for solids.

In summary, this work points out the incentives for using experimentally determined values for Henry's Law constant rather than the inaccurate calculated values. Additionally, many compounds are volatile that would not be judged to be if based on vapor pressure alone.

Removability

It was recognized, early in the research program that while treatability studies of pure compounds in distilled water were useful and provided needed information quickly, other studies which involved mixtures of compounds in wastewater solutions were equally important. Thus, concurrent with the pure compound research, pilot plant studies were undertaken to investigate the capabilities of selected treatment systems to remove mixtures of toxic compounds. The first phase of the study has been completed and is reported on below.(22) The research is being continued to include additional compounds and other treatment processes.

TABLE 19. HENRY'S LAW CONSTANTS

Priority Pollutant Number	Compound	Experimental	Calculated
		m^3 atm mol^{-1} x 10^{-3}	
1	Acenaphthene	0.241	solid
4	Benzene	5.55	5.49
6	Carbon tetrachloride	29.30	30.00
7	Chlorobenzene	3.93	3.71
8	1,2,4-Trichlorobenzene	1.42	2.31
9	Hexachlorobenzene	1.70	solid
10	1,2-Dichloroethane	1.10	1.34
11	1,1,1-Trichloroethane	4.08	4.92
12	Hexachloroethane	9.85	no data
13	1,1-Dichloroethane	5.45	5.53
23	Chloroform	3.39	3.23
25	1,2-Dichlorobenzene	1.94	2.00
26	1,3-Dichlorobenzene	2.63	2.97
27	1,4-Dichlorobenzene	2.72	solid
29	1,1-Dichloroethylene	14.90	15.10
30	1,2-Trans-dichloroethylene	5.32	4.05
32	1,2-Dichloropropane	2.82	2.31
33	1,3-Dichloroproylene	3.55	1.35
38	Ethylbenzene	6.44	6.44
44	Methylene Chloride	3.19	2.84
47	Bromoform	0.532	0.595
48	Bromodichloromethane	2.12	no data
49	Trichlorofluoromethane	58.30	136.00
51	Dibromochloromethane	0.783	no data
56	Nitrobenzene	0.0238	0.0230
60	4,6-Dinitro-o-cresol	0.0014	no data
65	Phenol	0.0013	solid
77	Acenaphthylene	0.114	solid
80	Fluorene	0.117	solid
85	Tetrachloroethylene	28.70	28.50
86	Toluene	5.92	6.44
87	Trichloroethylene	11.70	11.70
89	Aldrin	0.496	solid
90	Dieldrin	0.0584	solid
91	Chlordane	0.0479	solid
100	Heptachlor	1.48	solid
101	Heptachlor epoxide	0.0316	no data
107	Arochlor 1254	8.37	solid
113	Toxaphene	4.89	no data

In the pilot plant study, two treatment systems were operated - a physical-chemical system consisting of alum clarification and granular media filtration followed by granular activated carbon, the other was a conventional activated sludge system consisting of a primary settler, an aeration tank and a final settler. Raw wastewater was obtained from a large interceptor sewer serving a residential and commercial area of Cincinnati, Ohio. The feedwater to the pilot plants was spiked to contain approximately 150 µg/l of five selected organic compounds from the priority pollutant list - carbon tetrachloride, trichloroethylene, ethylbenzene, nitrobenzene and dimethyl phthalate. Samples were collected by automatic samplers (using the specially developed sampler for volatile compounds) and composited over three and four days. The volatile compounds were analyzed by a purge and trap method combined with gas chromatography while the non-volatiles were extracted with methylene chloride, solvent volume reduced and the analysis completed with gas chromatography with a flame ionization detector.

In addition to the five organics, four metals were spiked into the feedwater and removals monitored throughout the system. The metals included copper, lead, cadmium and mercury. In the case of both the organics as well as the metals, the initial concentration entering the treatment system was the sum of the spike and the background content of the raw wastewater.

Table 20 shows the removals obtained by alum clarification and dual media filtration. The first three compounds are volatile and one would expect the observed 40-60% loss during clarification. Some of the loss, however, was due to incorporation into the sludge.

TABLE 20. AVERAGE REMOVAL OF TOXICS BY ALUM CLARIFICATION AND FILTRATION

Compound	Raw ug/l	Raw-Spiked ug/l	Filtered ug/l	Removal %	Sludge mg/kg
Carbon tetrachloride	0	138	68	51	19
Trichloroethylene	0.1	169	102	40	60
Ethylbenzene	1.1	155	61	61	124
Nitrobenzene	38	170	109	36	311
Dimethyl phthalate	7.3	178	153	14	88
Copper	-	285	36	87	-
Cadmium	-	137	6	96	-
Lead	-	232	28	88	-
Mercury	-	19	1.1	94	-

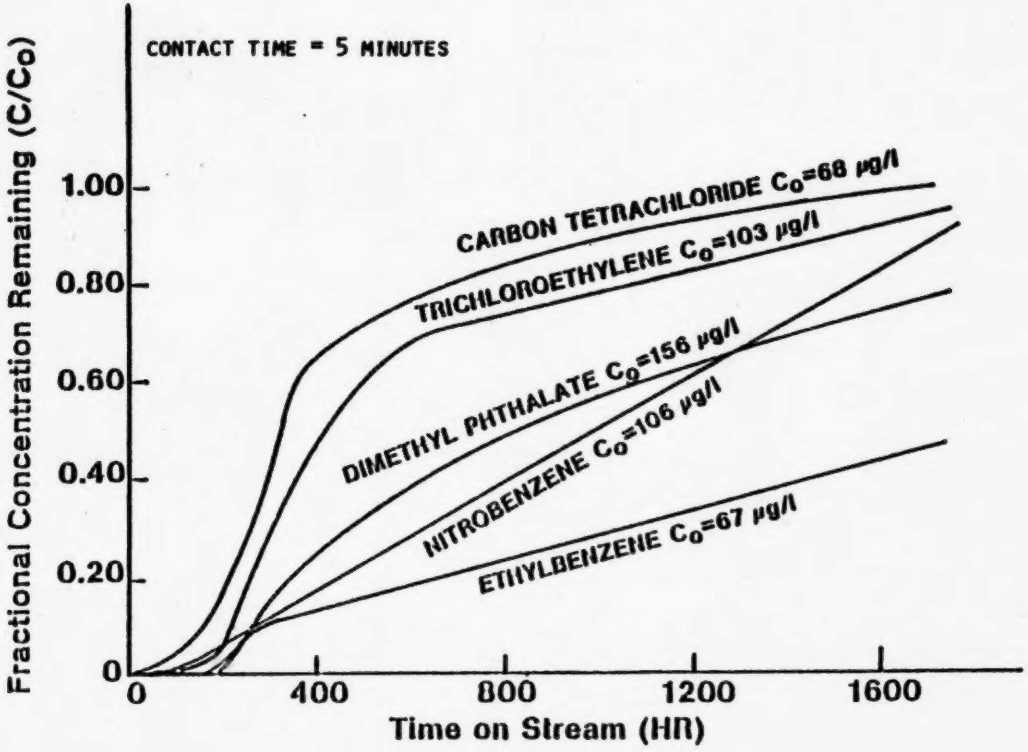
Not unexpectedly, good removals, ranging from 87-94%, were obtained for the metals. No sludge analyses are available at this time but it can be assumed that the amounts of metal removed are undoubtedly in the sludge.

The effluent from the alum clarification served as the feed to two granular activated carbon contactors, the first of which had an empty bed contact time of five minutes; the second provided ten minutes contact for a total of fifteen minutes. The purpose of the relatively short contact time in the first column was to obtain information on breakthrough within a reasonable time for all of the compounds tested. The breakthrough curves are shown in Fig. 6. The flow rate to the columns was 4.0 l/min (1.1 gpm). Almost total breakthrough was observed for carbon tetrachloride and trichloroethylene, which is consistent with the low carbon capacities determined by the isotherm treatability studies. Breakthrough was being approached for dimethyl phthalate and nitrobenzene while ethylbenzene had achieved a breakthrough of only 40% at the end of 1600 hours of operation. The breakthroughs cannot be rigorously explained by the isotherm data since the influent concentrations of the compounds reflected the varying removals by the prior alum clarification, and therefore, the feed concentrations varied from 67 µg/l to 156 µg/l for the compounds.

The data for the pilot activated sludge system are shown in Table 21. The feedwater was from the same source as that for the physical-chemical system. Removals for all of the organics were 98% or greater, resulting in small residual concentrations in the effluent. The mechanisms of removal are not readily evident from the data. It is clear, however, that carbon tetrachloride and trichloroethylene were probably removed from the system mainly by stripping, and that the remaining compounds were partially degraded and in part accumulated in the sludge. Removal of the metals ranged from 70-78% and while no sludge analyses are available, the metals removed from the bulk flow are almost certainly associated with the sludge.

TABLE 21. AVERAGE REMOVAL OF TOXICS BY ACTIVATED SLUDGE SYSTEM

Compound	Influent ug/l	Primary Effl. ug/l	Final Effl. ug/l	Removal %	Primary Sludge mg/kg
Carbon tetrachloride	130	119	0.8	99+	0.21
Trichloroethylene	182	164	2.3	99	4.7
Ethylbenzene	193	156	2.1	99	78
Nitrobenzene	173	167	3.1	98	158
Dimethyl phthalate	206	204	3.8	98	74
Copper	349	174	106	70	-
Cadmium	171	72	38	78	-
Lead	271	109	69	75	-
Mercury	20	7.4	6.	71	-



BREAKTHROUGH OF ORGANICS FROM 1.2m CARBON BED

FIGURE 6

SUMMARY

An attempt has been made in this paper to collect into one place information from many sources - legislative, regulatory, technical and investigative - in order to present a coherent view of the problems of toxic substances. The emphasis has been on those factors which most affect the municipal wastewater treatment plant.

Starting in the early 1970's, the U. S. Congress, through a series of legislative acts and amendments, has sought to control and regulate pollution in the environment. The Environmental Protection Agency is but one of several agencies which have been mandated a responsibility in the specific area of toxic substances. With the issue of the Consent Decree in 1976, and the designation of 129 specific compounds, research on the control, fate in the environment and health aspects of these and other toxic compounds began in earnest.

Much research is currently underway to respond to these needs. Survey and research data, however, are only now becoming available. Thus, this paper had had to rely on preliminary results - mostly unpublished - which are too sparse to form firm conclusions. Nevertheless, some tentative trends can be observed, which may or may not be supported when all of the data become available.

Thus it can be seen, from Table 3, that pretreatment of industrial wastewaters for removal of metals significantly reduced the concentration of metals in the raw wastewater to a municipal plant. This effect is most evident in the reduction of metals in the sludges.

Very early in the research, it became evident that analytical methods for the priority pollutants would have to be developed that were capable of qualitative and quantitative detection of these compounds at the parts-per-billion level in all types of wastewaters and sludges. This has been done, but it is clear that much more analytical methodology research will still be required.

With analytical procedures available, it was possible to start surveys of municipal plants - 65 cities in two separate surveys - to assess the situation in the field on the occurrence and concentration of priority pollutants entering and leaving municipal treatment plants, including the sludges. Based on the results of only three cities, it appears that most of the priority pollutants - 87 out of 129 - were detected in the influent but generally at low concentrations - < 10 ug/l. The treatment plants - activated sludge - were capable of achieving substantial reductions in concentrations for most of these compounds. Removals were apparently achieved by a combination of processes including stripping, biodegradation and by sorption into the sludge. Nevertheless, some thirty compounds.

were detected in effluent samples which were at or near the concentrations proposed for stream water quality criteria. While it can be expected that dilution in the surface waters will provide some margin for safety, these compounds may require some attention if other cities encounter higher influent concentrations. Studies of the emission of volatile priority pollutants from aerated grit chambers and aeration basins showed that, even at the liquid/air interface only one compound, trichloroethane, came close to reaching maximum permissible average air concentrations that have been established by OSHA. Clearly, much more data are needed before it can be concluded that emissions of priority pollutants from treatment systems either are or are not a problem.

In the short term, survey studies and research on removal of toxic organics by treatment systems will provide the answers to the immediate needs of the regulatory agencies. For the longer term, research on treatability of organic compounds is essential. It is from this type of research that the mechanisms of treatment - volatility, biodegradation, sorption, etc. - will be understood sufficiently in order to predict the behavior of organic compounds other than the limited list represented by the priority pollutants. Much research will be required on treatability to obtain the necessary biological-physical-chemical information in order to acquire the ability to predict the behavior of untested compounds.

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APPENDIX A

Federal Laws and Agencies Affecting Toxic Substances Control [a]

Statute	Year Enacted	Responsible Agency	Sources Covered
Toxic Substances Control Act	1976	EPA	Requires premarket evaluation of all new chemicals (other than food additives, drugs, pesticides, alcohol, tobacco); allows EPA to regulate existing chemical hazards not covered by other laws related to toxic substances
Clean Air Act	1970, amended 1977	EPA	Hazardous air pollutants
Federal Water Pollution Control Act	1972, amended 1977	EPA	Toxic water pollutants
Safe Drinking Water Act	1974, amended 1977	EPA	Drinking water contaminants
Federal Insecticide, Fungicide, and Rodenticide Act	1948, amended 1972, 1975, 1978	EPA	Pesticides
Act of July 22, 1954 (codified as §346(a) of the Food, Drug and Cosmetic Act	1954, amended 1972	EPA	Tolerances for pesticide residues in food
Resource Conservation and Recovery Act	1976	EPA	Hazardous wastes
Marine Protection, Research and Sanctuaries Act	1972	EPA	Ocean dumping
Food, Drug and Cosmetic Act	1938	FDA	Basic coverage of food, drugs and cosmetics
Food additives amendment	1958	FDA	Food additives
Color additive amendments	1960	FDA	Color additives
New drug amendments	1962	FDA	Drugs
New animal drug amendments	1968	FDA	Animal drugs and feed additives
Medical device amendments	1976	FDA	Medical devices
Wholesome Meat Act Wholesome Poultry Products Act	1967 1968	USDA	Food, feed and color additives and pesticide residues in meat and poultry
Occupational Safety and Health Act	1970	OSHA	Workplace toxic chemicals
Federal Hazardous Substances Act	1966	CPSC	"Toxic" household products (equivalent to consumer products)
Consumer Product Safety Act	1972	CPSC	Dangerous consumer products
Poison Prevention Packaging Act	1970	CPSC	Packing of dangerous children's products
Lead Based Paint Poison Prevention Act	1973, amended 1976	CPSC	Use of lead paint in federally assisted housing
Hazardous Materials Transportion Act	1970	DOT (Materials Transportation Bureau)	Transportation of toxic substances generally
Federal Railroad Safety Act	1970	DOT (Federal Railroad Admin)	Railroad safety
Ports and Waterways Safety Act Dangerous Cargo Act	1972 1952	DOT (Coast Guard)	Shipment of toxic materials by water

CPSC = Consumers Product Safety Commission
DOT = U.S. Department of Transportation
EPA = U.S. Environmental Protection Agency
FDA = Federal Drug Administration
OSHA = Occupational Safety and Health Administration
USDA = U.S. Department of Agriculture

[a] From "Environmental Quality," the Ninth Annual Report of the Council on Environmental Quality, December 1978.

APPENDIX B

KEY ENVIRONMENTAL LEGISLATION RELATED TO TOXICS MANAGEMENT AND MUNICIPAL WASTEWATER TREATMENT FACILITIES

Clean Water Act of 1977 (PL 95-217)	- provides for uniform, enforceable national standards for clean water, a national permit program for point-source discharges; federal funds for construction of sewage treatment systems, and state and area-wide planning and management programs to address nonpoint-source discharges.
201(g)	- construction grants
204(b)	- user charges
208	- management plans
301(h)	- relaxed effluent limits for municipal facilities
302(a)	- water quality based effluent limits
303	- water quality standards
304(a)	- publish and periodically update water quality criteria
304(g)	- pretreatment guidelines
307(a)	- toxic pollutant list & effluent limitations
307(b)	- industrial variances
307(b)(c)	- pretreatment standards
402(a)	- NPDES permits
403	- ocean dumping
405(d)	- issuance of guidelines for the disposal and utilization of sludge
405(e)	- requires conformance by POTW's to guidelines
Clean Air Act of 1977 (PL 95-95)	- provide for the protection and enhancement of air quality through regulation of pollutant emissions and establishment of air quality standards
112	- national emission standards for hazardous air pollutants (asbestos, beryllium, mercury, vinyl chloride, benzene, are currently regulated)

Appendix B (continued)

122	- certain unregulated pollutants such as cadmium, arsenic, polycyclic organic matter (soot), radioactive releases
Resource Conservation and Recovery Act of 1976 (PL 94-580)	- requires a regulatory system for the treatment, storage, and disposal of hazardous wastes
1008(a)(3)	- defining open dumping criteria
3001	- identifying hazardous municipal sludges and solid waste
3005	- permits for municipal facilities treating, storing or disposing of hazardous sludge on-site
4004(a)	- disposal facility criteria to distinguish sanitary landfills from open dumps
4005(b)	- inventory of open dumps
4005(c)	- prohibition of open dumping
Toxics Substances Control Act of 1976 (PL 94-469)	- gives EPA broad discretionary authority to control hazardous chemical substances (excludes pesticides). Provides for direct control of new and existing chemicals, requires premarket screening of new chemicals, and provides for authority to require testing of a chemical's toxicity. If an unreasonable risk is posed by a chemical the EPA can prohibit or limit its manufacture, processing, distribution, use and disposal.
Safe Water Drinking Act of 1974 Sec. 1412(a)	- Interim Primary Standards 9 inorganics, 6 pesticides, 3 categories of radioactive substances.
Federal Insecticide, Fungicide and Rodenticide Act of 1972 (PL 92-516)	- helps to control the entry of pesticides into the water environment.
Sec. 3	- All pesticides must be registered with and classified by EPA for general or restricted use.
Sec. 6	- Cancellation applied usually to a specific set of uses or inadequacy of a label.

APPENDIX C

CRITERIA FOR SLUDGE APPLICATION TO LAND(23)

Cadmium Criteria for Food Chain Crop Uses

1. Cd < 2 mg/kg - no special provision
   Cd > 2 mg/kg - pH 6.5 @ time of application

2. Annual application rate of cadmium - tobacco, leafy vegetables, root crops for human consumption 0.5 kg/ha

3. Annual application rate of cadmium - all other crops -
   2.0 kg/ha to June 30, 1984
   1.25 kg/ha to Dec. 31, 1986
   0.5 kg/ha to Jan. 1, 1987

4. Cumulative application rate for cadmium

Soil cation exch. cap. - meq/100g	Max. Cumulative Application rate (kg/ha)	
	Soil pH < 6.5	Soil pH > 6.5
5	5	5
5-15	5	10
>15	5	20
5	5*	
5-15	10*	
>15	20*	

* These application rates apply only if soil pH is maintained at pH 6.5 for food-chain crops or for animal feed where pH 6.5 is adjusted at time of application or planting, whichever occurs later and pH is maintained and a distribution plan for animal feed is prepared and notification of future land owners of the presence of high cadmium and the prohibition of food-chain crops.

Polychlorinated Biphenyls (PCB) Criteria

For animal feed uses including pasture crops for animals for milk production

PCB Content (dry weight)

<10 mg/kg	no special requirements
>10 mg/kg	sludge incorporation into soil or surface application and crop/milk monitoring for FDA criteria which are, 0.2 mg/kg act.vol. weight for animal feed crops, and 1.5 mg/kg (fat basis) for milk.

Disease Control Criteria	
Sludge applied or incorporated into the soil	Alternative 1 must be treated prior to application by a process to significantly reduce pathogens such as anaerobic or aerobic digestion, air drying, composting, lime stabilization or other similar technique and 12 months of controlled public access. One month prohibition of grazing animals whose products are consumed by humans. Alternative 2 burial or trenching
Septage Disposal	Alternative 1 must be treated prior to application by a process to significantly reduce pathogens such as anaerobic or aerobic digestion, air drying, composting, lime stabilization or other similar technique Alternative 2 site control provisions of 12 months of controlled access and one month prohibition of grazing animals whose products are consumed by humans. Alternative 3 burial or trenching
Where sludge or septage is applied to land where crops for direct human consumption are grown within 18 months of application	Process which Significantly Reduces Pathogens (PSRP), in addition, processes to further reduce pathogens such as beta ray irradiation, gamma ray irradiation, pasturization or other equivalent methods or processes such as high temperature composting, heat drying, heat treatment or thermophilic aerobic digestion with site control provisions if there is direct contact of sludge and edible portion of the crop.

GROUNDWATER PROTECTION CRITERIA

Maximum Contaminant Level (MCL) for Inorganic Chemicals	Level (mg/l)
Arsenic	0.05
Barium	1.0
Cadmium	0.010
Chromium	0.05
Lead	0.05
Mercury	0.002
Nitrate (N)	10.0
Selenium	0.01
Silver	0.05
Fluoride (function of temperature)	1.4-2.4

MCL for Organic Chemicals[1] Chlorinated Hydrocarbons	Level (mg/l)
Total Trihalomethanes	0.10
Endrin	0.0002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005

Chlorophenoxys	Level (mg/l)
2,4-D (2,4-dichlorophenoxy-acetic acid)	0.1
2,4,5-T Silvex (2,4,5-trichlorophenoxy-proprionic acid)	0.01

1. These are the currently regulated organic chemicals. The Agency is considering the establishment of MCL's for the following additional organic chemicals in the μg/l to mg/l range:

Trichloroethylene
Tetrachloroethylene
Carbon tetrachloride
1,2-Dichloroethane
1,1,1-Trichloroethane
1,1,2-Trichloroethane
cis-1,2-Dichloroethylene

HEALTH EFFECTS RESEARCH ASSOCIATED WITH MUNICIPAL WASTEWATER TREATMENT AND SLUDGE DISPOSAL

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INTRODUCTION

Many wastewater treatment plants are constructed in urban areas close to residential areas. These systems generally contain aeration basins or trickling filters where there is an opportunity for small droplets of wastewater to be emitted. These droplets could contain a bacterium or virus and evaporate very rapidly to yield droplet nuclei in the size range of 1 to 20 microns in diameter, known as aerosols.

A number of persons have investigated the types of organisms emitted and the density in relation to distance from the treatment plant. Probably the best overview of the subject was written by Hickey and Reist (1,2). These microorganisms travel passively with the wind and their density decreases with time and distance as a result of atmospheric dispersion, die-off (loss of viability), and deposition (removal from the air) (3). The potential for plant workers and nearby residents to inhale viable organisms certainly exists. However, there has never been a systematic investigation to confirm or negate the existence of a health hazard from these viable wastewater aerosols.

With this background, the Health Effects Research Laboratory of the U.S. Environmental Protection Agency arranged for several studies to gather information on health effects associated with aerosols from uncovered wastewater treatment plants. These studies were conducted by personnel from universities or research institutions.

EPIDEMIOLOGY STUDIES

Two types of epidemiological studies were carried out. The first type involved the analysis of existing public health data to determine if there was any relationship between the operation of a wastewater treatment plant and disease patterns. These retrospective studies could be carried out quickly because the basic data already existed and just needed to be analyzed statistically. The second type of investigation consisted of prospective seroepidemiological studies, involving blood samples for antibody testing for various microbiological agents, as well as clinical samples, such as throat cultures or stool samples.

Retrospective Studies

The first investigation utilized data available from a community health study which was carried out over a seven-year period at Tecumseh, Michigan (4). Illness in the community was related to distance from the activated sludge plant in Tecumseh. Dwelling units around a nonemitting location upwind from the treatment plant were used as a control group. Results showed that both respiratory and gastrointestinal illnesses were greater than expected for persons living within 600 meters of the sewage treatment plant. However, the data suggest the higher rates are related to higher densities of lower socioeconomic families with crowded living conditions rather than to the treatment plant (5).

In a second retrospective study (6), illness rates of students at an elementary school, located adjacent to a new advanced wastewater treatment plant, were measured by analyzing recorded absenteeism at this school before and after start-up of the new plant. Attendance at eight nearby schools was used for control rates. There were two sources of aerosols at the treatment plant, the aeration basin of the secondary treatment process and an aerated surge basin used to equalize the flow into the secondary process. Aerosol exposure locations for the children could take place at two locations, the playground area which was within 50 meters of the aerated surge basin, and the classroom area located 400 meters from the aeration basin. On warm days the classroom windows were open.

A daily exposure index was computed for each location-aerosol source combination, based on the four wind direction observations for the day recorded at a nearby airport. If an observation was within the range of downwind directions for the location-source combination, it was given a value of 1.0. Wind direction observations within 30 degrees of this range were considered to represent occasional exposure and given a value of 0.5. The four observation values for the day were then summed to yield the daily exposure index for that location-source combination. Thus, a daily exposure index of 4.0 would indicate steady exposure from the aerosol source throughout the school day, while a daily exposure index of 0.0 would indicate no exposure.

The frequency distribution of the daily exposure index over the 355 school days, over a two year period following operation of the treatment plant, are presented in Table 1. A review of the data indicates that the student exposure was relatively infrequent.

Densities of microorganisms were measured in the aerosols at 30 and 100 meters downwind from the aeration basins and projected to the distance at the school. Based on breathing rates, a child would inhale a maximum of 3.5 fecal streptococci and 9 mycobacteria during a day when the wind blew steadily toward the school.

Detailed examination of school attendance patterns for the two year period prior to the operation of the sewage treatment plant and the two year period following operation revealed no difference for the school next to the treatment plant. Similar comparisons of attendance at this school with the eight other nearby schools showed no differences. Although school attendance is not the most accurate indicator of illness, the data provide no evidence of an adverse health response from the treatment plant.

TABLE 1. FREQUENCY DISTRIBUTION OF DAILY EXPOSURE INDEX OF STUDENT EXPOSURE

	Number (N) and Percent (%) of School Days							
Exposure Location:	Classroom Area				Playground Area			
Aerosol Source:	Aeration Basin		Surge Basin		Aeration Basin		Surge Basin	
Daily Exposure Index	N	%	N	%	N	%	N	%
0.0	196	55	191	54	155	44	137	39
0.5	48	14	53	15	58	16	52	15
1.0	35	10	31	9	45	13	44	12
1.5	22	6	27	8	23	6	26	7
2.0	21	6	26	7	23	6	34	10
2.5	13	4	13	4	18	5	21	6
3.0	10	3	9	3	17	5	14	4
3.5	4	1	2	1	8	2	10	3
4.0	6	2	3	1	8	2	17	5

Prospective Studies

The first seroepidemiological study was carried out around a new 64,000 cubic meter per day activated sludge plant in the State of Illinois (7). Persons were evaluated for one year prior to start-up of the treatment plant and one year after operation.

The epidemiological studies were divided into two parts. One part consisted of having over 4200 persons who lived between 0.4 and 5.0 kilometers from the plant answer questions in a questionnaire regarding their illnesses and health status. Detailed statistical analyses were then conducted to determine if results could be related in any way to the operation of the plant. Another part of the study included taking clinical specimens from 282 persons near the plant. Table 2 presents the type of tests made on the human subjects.

TABLE 2. CLINICAL TESTS OF PARTICIPANTS

Blood - metals, antibodies
Urine - metals
Hair - metals
Throat swabs - bacteria, virus
Feces samples - bacteria, virus, metals,
helminths, protozoa

Environmental measurements near the treatment plant indicated that it was a source of indicator bacteria, coliphage, specific pathogenic bacteria, enteroviruses, and mercury in the aerosols emanating from the aeration basins. However, the levels of microbiological or chemical agents of the air, soil, and water samples in the neighboring residential areas were not distinguishable from the background levels.

Based on questionnaire results alone, the responses showed a slight increase in gastrointestinal symptoms after operation of the plant for persons living the nearest distance. Streptococci isolations in the throat swabs also increased. In contrast, tests for 31 viral antibodies and attempted isolations of many pathogenic bacteria, parasites, and viruses yielded no evidence of an adverse health effect from the wastewater treatment plant. On balance, the overall conclusion was that the plant did not present a health hazard for nearby residents.

Another seroepidemiological study was conducted near a one million cubic meter per day sewage treatment plant in Chicago, Illinois (8). The surface area of sewage exposed to the atmosphere in aeration basins and settling tanks is 55,000 square meters. The plant has been in operation since 1929.

Study design was somewhat similar to that described above. A health questionnaire survey was made of 2,378 persons over an eight month period. Stool and throat specimens were collected from children under age 12 for bacteria and virus isolates. A total of 318 persons gave paired blood samples, at the beginning and end of the eight month period, which were tested for antibody to three types of polioviruses, five types of coxsackie B viruses, and four types of echoviruses. Antibody titers in the respective sera obtained at the beginning of the study were used to determine the rates of previous infections by these specific viruses and the number of persons susceptible to each of these agents. The air near the treatment plant was sampled for certain gases, metals and viable particles which could be emitted by the plant.

Table 3 shows that the plant was a source of total viable particles and total coliforms. The coliforms were at background levels at 800 meters downwind while total viable particles were still above background at 800 and 1600 meters.

Table 4 relates the number of persons whose paired blood samples showed an increase in antibody level for any of the five coxsackieviruses and four echoviruses tested, compared to the average household exposure to total viable particles for these persons. Negative seroconversion data are also presented. The differences observed were not sufficient to suggest the seroconversions were related to high exposure to microbial aerosols, as indicated by total viable particle counts. In fact, persons who seroconverted to echoviruses were, on the average, exposed to lower counts.

Overall conclusions from this investigation were that the sewage treatment plant was a source of airborne total aerobic bacteria-containing particles and total coliform bacteria as measured in the community. The concentration profiles of trace metals, particulates, nitrates, sulfates, sulfur dioxide, nitrogen dioxide, hydrogen sulfide, and chlorine in the community indicated that the plant was not a source of these constituents. Relationships were not

TABLE 3. AVERAGE DENSITIES OF MICROBIAL AEROSOLS

Distance	Total coliforms number/m^3	Total viable particles number/m^3
0.8 km upwind	1.2 (26)*	143 (62)
On-plant downwind	6.9 (26)	376 (68)
0.8 km downwind	1.2 (25)	198 (68)
1.6 km downwind	1.0 (24)	218 (60)

* Number of samples shown in parentheses.

TABLE 4. SEROCONVERSIONS COMPARED TO EXPOSURE

Virus Type	Average total viable particle exposure, number/m^3
Coxsackieviruses B1 to 5	
Seroconversion (46)*	175
No seroconversion (272)	160
Echoviruses 3, 6, 9, 12	
Seroconversion (25)	145
No seroconversion (292)	163

* Number of persons in category.

found by regression analyses between total viable particles exposure indices and:

(a) self-reported acute illness rates,
(b) pathogenic bacteria isolation rates,
(c) prevalence rates of antibody to certain enteroviruses, and
(d) virus antibody titers.

In an effort to check persons with the highest exposure to wastewater aerosols, and with the best probability of showing a response, studies were made of wastewater workers. It is recognized that such exposure could be due to direct contact with sewage or handling equipment contaminated by sewage as well as exposure by means of the aerosol route. There have been a few efforts in the past to evaluate the health of sewage treatment plant workers, but most of these studies were of limited scope (9-17).

A prospective seroepidemiological study was carried out in three metropolitan areas: Cincinnati, Ohio; Chicago, Illinois; and Memphis, Tennessee (18). The primary study group consisted of about 100 newly employed activated sludge plant workers at the three cities. In addition, in Cincinnati two other groups of about 50 persons each were evaluated, sewer maintenance workers and persons who had worked at a primary sewage treatment plant prior to the activated sludge plant work. As additional control groups, water treatment plant workers were selected in Chicago, utility workers in Memphis, and highway maintenance workers in Cincinnati.

Detailed medical tests were conducted on both exposed workers and controls. Factors considered included liver function, immunoglobulin levels, illness rates, blood chemistries, urinalysis, parasitic, bacterial, and viral infections, and antibody to several bacteria and numerous viruses. A serologic survey for viral antibodies of the families of study participants was also conducted to check on the possibility of transmission of infectious agents from the job to the home, and consisted of 87 families of wastewater workers and 41 control families.

The only difference in illness rates among worker groups which was statistically significant was the difference in gastrointestinal illness between inexperienced workers and other groups. Results between experienced workers and controls were not significantly different. This is shown in Table 5. These gastrointestinal illnesses were mild and generally appeared within the first several months of employment. There was no relationship between the occurrence of illness and rise of antibody titer for any of the numerous agents checked.

The study failed to demonstrate any differences among groups for immunoglobulin levels or liver function tests. There was no evidence of any increase in hepatitis, as reflected by the liver tests, in workers exposed to wastewater. Stool examinations showed no evidence of increased parasitic infection in the wastewater exposed workers. Isolations of Salmonella and Shigella in rectal swabs were not significantly different between the exposed and non-exposed groups, nor were blood antibody to Salmonella, Leptospira and Legionella pneumophila different. Interestingly, 34.4 percent of the workers exposed to

TABLE 5. CLINICAL ILLNESS IN WASTEWATER WORKER GROUPS

	Illness per 100 Worker-months of Exposure				
Worker Group	Respiratory	Gastro-intestinal	Combined Respiratory and Gastro-intestinal	Other	Total
Inexperienced	6.5	3.6	1.7	3.8	15.6
Experienced					
Treatment Plant	8.0	2.4	0.9	3.6	14.9
Sewer Maintenance	5.5	1.3	0.5	4.0	11.3
Control	4.9	1.5	1.0	3.3	10.7

sewage and 35.4 percent of the controls had antibody to Legionella pneumophila. The possibility of an increase in viral infections in wastewater workers was also investigated by means of cultures of throat and rectal swabs and extensive serologic surveys. A major concern in this study was the occurrence of enterovirus infections because of the frequency that enteroviruses can be recovered from sewage. Neither the serologic tests for antibody nor culture results demonstrated a significant increase of viral infections in the wastewater exposed workers. In the study of families, there were no differences between results of the two groups of families.

AEROSOL ASSESSMENT

The comprehensive investigations described leave little doubt that working at or living near a wastewater treatment plant does not present a microbiological hazard to the people. However, it should always be kept in mind that sewage contains potentially pathogenic agents and workers at sewage treatment plants should always maintain good personal hygiene and sanitation practices. Washing hands with soap and water prior to eating and changing from work clothes to street clothes at the end of the work day are two examples of good practice.

When sewage treatment takes place within a totally enclosed building, the treatment plant workers are potentially exposed to higher concentrations of infectious agents, allergens or irritants than in an outdoor setting. Although one study of employee health in enclosed sewage treatment facilities in Manitoba, Canada did not demonstrate unequivocally a relationship between pathogens in the work environment and employee illnesses, short-term absenteeism due to sinusitis seemed to be related to the work environment (19). Adequate ventilation and a monitoring program for airborne contaminants and employee health should be encouragedin this situation.

One may wonder why, when people have been exposed to microorganisms from the wastewater treatment processes, they did not generally become infected or ill. A combination of several reasons may be offered.

1. The densities of specific pathogens in the aerosols were low, and were reduced rapidly with time and distance from the source.
2. With a respiration rate of approximately one cubic meter per hour, a person would ordinarily inhale very few organisms unless constantly exposed for many hours.
3. The exposure levels were below the minimum infective dose.
4. Microorganisms in wastewater are primarily enteric organisms whereas the route of exposure was respiratory. The proper surface receptor sites for the organisms may not have been available.
5. Nonspecific immunity, which responds quickly to foreign substances, was capable of handling the few microorganisms inhaled. In this situation phagocytic cells, called macrophages and polymorphonuclear leukocytes, tend to engulf the particles from the external environment and subsequently digest them. These actions undoubtedly took place prior to initiation of the specific humoral and cell-mediated immunity response whereby specific antibody and T-cells are formed.
6. Exposed persons were probably in good health and better able to combat infection than would sickly persons.
7. Infectious agents other than those checked may have caused an undetected infection.

MUNICIPAL SLUDGE RESEARCH PROGRAM

In addition to research on wastewater treatment, the Health Effects Research Laboratory has undertaken a research program on potential health problems encountered when municipal sewage sludge is applied to land for agricultural purposes. Several of our studies are conducted jointly with the Municipal Environmental Research Laboratory. The objective is to provide a scientific basis for determining the extent to which the U.S. Environmental Protection Agency should regulate agricultural uses of municipal sewage sludge. It is my intent to briefly outline the types of projects being carried out this year.

Cadmium

Of the various trace metals that may be contained in municipal sludge, cadmium is the metal of most concern. The studies may be put into four groups: background information, fundamental studies, high risk populations, and assessments.

Tables 6, 7 and 8 list the types of studies underway in the first three groups. Assessments are carried out by U.S. EPA staff and attempt to bring all the information together in a manner which allows a judgment to be made on maximum permissible levels in sludge.

TABLE 6. BACKGROUND TASKS ON CADMIUM

1. Cadmium and beta$_2$-microglobulin levels in urine of the United States population.
2. Cadmium and other metals in soils and food crops of the United States.
3. Cadmium intake as measured by stool samples.
4. Passage of metals, organics and pathogens through the sludge-soil-plant-animal pathway.

TABLE 7. FUNDAMENTAL STUDIES WITH CADMIUM

1. Mechanism of intestinal cadmium absorption in rats.
2. Availability of cadmium to rats from crops grown on cadmium enriched soils.
3. Role of aging processes on chronic effects in the rat kidney from cadmium.

TABLE 8. STUDIES OF HIGH RISK GROUPS

1. Assessment of cadmium exposure and risk in an American vegetarian population.
2. Cadmium intake and health effects in a population with a high oyster consumption.
3. Ingestion and absorption of heavy metals in undernourished populations.

Microbiological

Since municipal sludge contains many bacteria, viruses, and parasites, there is interest by the U.S. EPA of potential microbiological hazards associated with the use of the sludge on agricultural lands. Table 9 summarizes several health related studies being conducted. With a valid data base, the Agency may issue regulations for the proper handling of sludge when it is planned to utilize its nutrient value for agricultural lands.

TABLE 9. MICROBIOLOGICAL PROJECTS RELATED TO SLUDGE

1. Health risks associated with composting sludge.
2. Microbial aerosols generated with spraying liquid digested sludge by tank truck.
3. Epidemiological study of farm families that utilize municipal sludge.
4. Assessment of health risks from microorganisms in sludge applied to land.
5. Survival of Ascaris eggs in sludge applied to soils.
6. Survey of sludges for parasitic contamination.

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UNITED STATES PAPERS PRESENTED AT TECHNICAL SEMINAR
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SECTION 301(h) OF THE CLEAN WATER ACT OF 1977 - A MID-COURSE CORRECTION IN MARINE POLLUTION CLEAN-UP........................ 721
Henry L. Longest, II, Office of Water Program Operations, ORD, USEPA

IMPROVED OPERATION AND MAINTENANCE OPPORTUNITIES AT MUNICIPAL TREATMENT FACILITIES.. 731
John M. Smith, Francis L. Evans III and Jon H. Bender, Municipal Environmental Research Laboratory, ORD, USEPA

STATUS OF DEEP SHAFT WASTEWATER TREATMENT TECHNOLOGY IN NORTH AMERICA.. 777
Richard C. Brenner and John J. Convery, Municipal Environmental Research Laboratory, ORD, USEPA

CURRENT DESIGN AND OPERATING EXPERIENCE WITH ANAEROBIC SLUDGE DIGESTION.. 825
Walter E. Garrison, Carl A. Nagel and R. Steven Easley, Los Angeles County Sanitation Districts, Whittier, California

SLUDGE COMPOSTING: PROCESSES AND FUTURE DIRECTIONS............ 863
Atal E. Eralp and Joseph B. Farrell, Municipal Environmental Research Laboratory, ORD, USEPA

SECTION 301(h) OF THE CLEAN WATER ACT OF 1977
A MID-COURSE CORRECTION IN MARINE POLLUTION CLEAN-UP

by

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Section 301(h) of the Clean Water Act of 1977 is, perhaps, the most important provision of this Act. It is important not only to the United States, but to Japan and to all nations that utilize the resources of our oceans. Briefly, Section 301(h) provides for the modification of secondary treatment requirements for publicly owned treatment works (POTWs) which discharge into marine waters, if they can demonstrate, to the satisfaction of the Administrator of EPA, compliance with eight statutory criteria established by Congress. Before I get into detail about Section 301(h), let me give you a few facts to point out the importance of this provision to Japan and some historical background on U.S. water pollution control legislation to put Section 301(h) into perspective.

Japan which is about the size of the State of California has a coastline of 18,425 miles, well over half the length of that of the United States. This fact, together with excellent oceanographic fishing conditions has contributed to make Japan one of the world's leading fishing nations. A recent annual report of the Food and Agriculture Organization of the United Nations showed that Japan ranked first in fishing production and in 1977 Japan's total fish catch was 10.76 million tons. 1/ dependence on the sea, Japan has become increasingly concerned about the pollution of ocean waters. In the last 35 years, as Japan has emerged as an industrial power, it has experienced the same problems as other industrialized nations - increased pollution from large industrial complexes and from concentrated population centers both of which are primarily located in coastal areas. In a report by the Japanese Ministry of Foreign Affairs, it was reported that Japan has a total of 12 petrochemical complexes in operation - all located along coastlines. 2/ At the same time, the rapid growth and concentration of population in urban centers in coastal areas has added to the environmental stress on coastal waters, particularly those of bays and inland seas.

I point out these facts simply to show how similar the problems are that our countries are facing, and to stress the fact that what we do with respect to the oceans could have profound effects on all of us. Senator Edmund Muskie, the Chairman of the U.S. Senate Subcommittee on Environmental Pollution, made this point quite clearly when he said:

> Our planet is beset with a cancer which threatens our very existence and which will not respond to the kind of treatment that has long been prescribed in the past. The cancer of water pollution was engendered by our abuse of our lakes, streams, rivers, and oceans; it has thrived on our half-hearted attempts to control it; and like any other disease, it can kill us. 3/

With these strong words, Senator Muskie introduced one of the most important and controversial pieces of environmental legislation in the United States and, perhaps, the world. The Federal Water Pollution Control Act Amendments of 1972 (Public Law 92-500) commonly referred to as the Clean Water Act, was born in an environment of controversy which has surrounded it throughout its existence. Vetoed by President Nixon on the grounds that it carried "an unconscionable price tag", the legislation came close to not being enacted into law. However, a unified Congress felt strongly enough about the threats of continued and worsening water pollution, to override the President's veto by an overwhelming margin. This action by Congress did not end the controversy.

Another controversial aspect of this legislation was that it contained a major change in the enforcement mechanism of the Federal water pollution control program - a shift of emphasis from water quality standards to technology-based effluent limitations. Both of these controversial aspects of the 1972 legislation are intimately related to Section 301(h) of the Clean Water Act of 1977. This Act, like the 1972 Act, also amended the Federal Water Pollution Control Act.

Prior to 1972, U.S. water pollution control laws established clean-up requirements for dischargers based on their impact on receiving water quality. Water quality standards were to be set as the control mechanism. 4/ Under this approach, individual States were to designate the uses of water to be protected, the kinds and amounts of pollutants to be permitted, the degree of pollution abatement to be required, and the time in which such abatement should be achieved.

It quickly became apparent that there were a number of problems inherent in this approach to clean-up. Difficulties were encountered in establishing reliable and enforceable effluent limitations on the basis of a given stream quality. This resulted from the imprecision of models for water quality and the effects of effluents in most waters. Also, the fact that water quality standards relied on the assimilative capacity of receiving waters was viewed as a problem. Thus, the Federal Water Pollution Control Act Amendments of 1972 (the 1972 Act) introduced a completely different approach to water pollution control - minimum nationwide requirements based on the capabilities of treatment technology. Under this approach, similar dischargers were required to clean up to the same extent, regardless of their impacts on the receiving waters. In addition to these nationwide requirements, the 1972 Act introduced a nationwide permit system called the National Pollutant Discharge Elimination System (NPDES), and authorized a major funding commitment by the Federal government for construction of municipal treatment works. The permit system provided an effective method for enforcement of effluent limitations and other conditions which were contained in the permits. The Administrator of EPA was given the authority to enforce those limitations and conditions under Section 309 of the Act. The virtues of this approach were clear. It made detection of violations and subsequent enforcement easier, and it was equitable.

The 1972 Act established effluent limitations for "point sources" which was defined under the Act, and it distinguished "publicly owned treatment works" (POTWs) from other point sources. For POTWs, the 1972 Act established two clean-up milestones. First, every POTW had to achieve effluent limitations based on "secondary treatment" by not later than July 1, 1977, and, by not later than July 1, 1983, effluent limitations based on the "best practicable waste treatment technology." EPA, subsequently, defined secondary treatment in terms of four pollutant parameters - biochemical oxygen demand (BOD), suspended solids, pH and fecal coliform - and established national uniform minimum effluent limitations for these pollutants to be attained by all POTWs by the 1977 deadline. (fecal coliform limitations were deleted from the definition in 1976).

With this background, I should explain the relationship between the two controversial aspects of the 1972 Act, to which I referred earlier, and POTWs. First, the "unconscionable price tag" of the 1972 Act was made up primarily of 18 billion dollars that was to be used for federal funding of up to seventy-five percent of the costs of construction of municipal waste treatment plants, i.e., POTWs. Secondly, the uniform secondary treatment requirements, almost immediately became the subject of attack by a number of West Coast municipalities which argued to both Congress and EPA that secondary treatment was not necessary to protect the marine environment or to assure the attainment and maintenance of water quality in deep ocean waters. Thus, the controversy continued. On the one hand, the President, concerned about the high costs of clean-up, impounded a substantial portion of the money allocated for the job, while, on the other hand, debate raged over the merits of secondary treatment for certain POTWs discharging into marine waters.

The municipalities which argued against secondary treatment contended that it traditionally had been defined in terms of pollutant parameters and levels of pollutant reduction which are important for fresh water ecology, where the discharge of oxygen-demanding wastes and sedimentation of suspended solids results in distinct environmental degradation, but that such pollutants have little significance for deep ocean waters where wastes are rapidly aerated and dispersed by strong currents and tidal actions. Some POTWs located in West Coast estuaries which exhibit a high degree of flushing also argued that secondary treatment provided no significant environmental benefit because discharges were rapidly oxygenated, dispersed, and carried into the open ocean. On this basis, these municipalities maintained that they should be exempted from the 1972 Act's secondary treatment requirement, and the associated capital, maintenance and operating costs.

In response to this debate over secondary treatment, EPA, in 1974, formed the Task Force on Secondary Treatment of Municipal Ocean Discharges. This task force was asked to examine the need for secondary treatment of wastewater discharged to the ocean from POTWs and to make recommendations on whether to propose to Congress any exceptions to the secondary treatment standards. Additionally, the National Commission on Water Quality which had been established under the 1972 Act to study and report to Congress on all aspects of achieving or not achieving the 1983 effluent limitations and goals established in Section 301(b)(2) of the Act, made recommendations concerning marine outfalls in its Reports to Congress in March, 1976. The recommendations of the two groups differed.

The EPA Task Force concluded that any amendments to the 1972 Act should maintain the concept of technology based effluent limitations, because not enough was known about the effect of pollutants on the ocean environment to permit the establishment of effective guidelines or protocols for a case-by-case approach to establishing effluent limitations. 5/ It recommended that in some cases the July, 1977 deadline should be extended so that further study could be done on new, undeveloped technologies which might prove better suited to marine pollution control than secondary treatment. 6/

The National Commission on Water Quality staff also noted the lack of information that existed on the impacts of pollution on marine ecology. 7/ However, the Commission, in its report, recommended that:

> Congress authorize waiving, deferral or modification of the 1977 requirements on a category-by-category basis for near-shore ocean discharges of publicly owned treatment works... where the Administrator determines that the adverse environmental impacts of such action will be minimal or nonexistent... 8/

Despite this recommendation there was substantial reservation about wholesale revision of either the 1977 treatment requirements or the concept of uniform technology based standards. Nevertheless, as a result of the testimony of the West Coast municipalities before Congress, the Congress included Section 301(h) in the Clean Water Act of 1977.

As I mentioned earlier, Section 301(h) provides that the Administrator of EPA, upon application by a POTW and with the concurrence of the State, may issue a NPDES permit which modifies EPA's effluent limitations for BOD, suspended solids and pH, where the applicant discharges into certain ocean and estuarine waters and demonstrates to the satisfaction of the Administrator, by showing compliance with eight statutory criteria set forth in Sections 301(h)(1) through 301(h)(8), that the modification will not result in any increase in the discharge of toxic pollutants or otherwise impair the integrity of receiving waters. The permits containing these modified requirements are to be issued for a fixed term not to exceed five years. Decisions on reissuance or termination of the permits will have to be made at the time of expiration.

At this point, the obvious questions are what are these statutory criteria and what does Section 301(h) do to the technology-based effluent limitations scheme established by the 1972 Act. To answer the latter question, it is necessary to consider the history of the 1977 amendments. In introducing the Conference Report on the 1977 amendments, Senator Muskie noted that, Congress, in the 1972 Act, established as an objective the restoration of the chemical, physical, and biological integrity of the Nation's waters, and a national goal of the elimination of the discharge of pollutants. 9/ He hastened to add that, with the 1977 Act, those objectives and goals remained the same.

There was a general consensus that the 1972 Act was working, that the Nation's waters were being cleaned up. The 1977 Act was not intended to make widespread changes to the national water pollution clean-up effort. Rather, it was viewed as an effort at "fine tuning" the clean-up program and as a "mid-course" correction to be made in light of lessons learned and experience gained after 5 years of effort. Thus, Section 301(h) was viewed as a part of that fine tuning and the eight statutory criteria that it set forth were intended to assure continued progress in protecting and improving the Nation's marine waters, while, at the same time, providing flexibility to direct limited resources to priority water pollution problems.

To answer the first question, the eight statutory criteria which an applicant must meet are the following:

First, there must be an applicable water quality standard specific to the pollutant for which the modification is requested. The degree of effluent reduction necessary to meet this standard must be provided as a minimum.

Second, the modified requirements must not interfere with the attainment or maintenance of that water quality which assures protection of public water supplies and maintenance and propagation of a balanced indigenous population of shellfish, fish and wildlife, and must allow recreational activities in and on the water.

Third, the applicant must have established, to the extent practicable, a system for monitoring the impact of such discharge on a representative sample of aquatic biota.

Fourth, the modified requirements must not result in any additional requirements on any other point or nonpoint source.

Fifth, all applicable pretreatment requirements for sources introducing waste into such treatment works must be enforced.

Sixth, the applicant, to the extent practicable, must have established a schedule of activities designed to eliminate the entrance of toxic pollutants from nonindustrial sources into such treatment works.

Seventh, there must not be any new or substantially increased discharges from the point source of the pollutant to which the modification applies above that volume of discharge specified in the permit.

The last criterion is that any Title II Construction Grant funds available to the owner of the treatment works must be used to achieve the degree of effluent reduction required by Section 201(b) and (g)(2)(A) or to carry out the requirements of Section 301(h). The two sections of the Act referred to in this last criterion call for the application of the best practicable waste treatment technology over the life of the POTW. These criteria placed a substantial burden on POTWs seeking modifications under Section 301(h). This was Congress' way of showing that they were seriously committed to the objectives and goals of the Act.

EPA promulgated regulations implementing Section 301(h) in June, 1979. Again, this was not an easy task, nor was it accomplished without controversy. I should point out that EPA, like all Federal agencies, is required by law to provide public notice and opportunity for comment to the general public on all rule-making by the Agency. Additionally, EPA, as a matter of policy, encourages public participation above and beyond the requirements of law. Thus, to assure adequate public participation the Agency held a public meeting to obtain comments on how the statutory criteria should be implemented, and then, after proposed regulations were published, public hearings were held to receive comments on them. In addition, many written comments on the regulations were submitted.

As you can imagine, there were many diverse opinions on how Section 301(h) should be implemented. Rather than dwell on these differences, I shall simply point out a few of the major areas of dispute.

Perhaps the most controversial aspect of the regulations was the agency's interpretation of the term "existing discharge" which appears in Section 301(h) and the limitation which that interpretation placed on eligibility for a modification. The Agency originally interpreted this to mean that Section 301(h) modifications could only be obtained for those discharges which existed on the date of enactment of Section 301(h), i.e., December 27, 1977, and that modifications could not be based on any changes to those discharges - such as relocation of outfalls, improvements to outfalls or treatment works - even if they had been planned for some time or were currently under construction. Needless to say, many commenters claimed that this interpretation contravened Congress' intent. In response to the public comments, the Agency reexamined the legislative history and concluded that, indeed, Congress did intend to allow modifications to be based on improvements.

A second controversial issue was the amount of data gathering and the uniform reporting requirements, particularly with respect to small, remote communities which discharged primarily domestic waste having little or no toxic materials. In response, EPA developed a separate policy for Native Alaskan Villages and indicated it would use its discretion in scheduling secondary treatment for these villages and communities in Puerto Rico and the U.S. territorial possessions in the Caribbean and the Pacific, where industrial toxic wastes were not a factor. At the same time, the Agency pointed out that it had no authority to categorically exempt communities based on the size or volume of their discharge, but that given the nature of their discharges, the monitoring and reporting requirements should be substantially reduced for them.

Finally, the difficulty of predicting the impacts of future outfalls or treatment system improvements was much commented upon. This was raised in the context of whether it was possible to make predictive judgments as to whether future construction would enable an applicant to meet the stringent water quality, physical, chemical and biological criteria set forth in the regulations, and would enable applicants to show that their discharges would assure the protection and propagation of a balanced, indigenous population of shellfish, fish and wildlife. The determination of what constitutes a balanced, indigenous population is, in itself, very difficult. In response to the comments on this issue, the Agency advised applicants of the difficulty of this task and of the lack of established methodologies for making such predictions, but concluded that applicants would not be excluded from applying for modifications simply because their applications required predictive judgments.

When the final regulations were published they included a deadline for submission of applications which gave POTWs ninety-days to submit their applications. This deadline occurred on September 13, 1979, and, while it also is a subject of dispute, seventy final applications were submitted as of that date. Due to the relatively recent deadline for submission of applications, the final results on the impacts of Section 301(h) are not yet determined. The Agency is currently reviewing the applications that have been submitted, but it is too early to predict which or how many applicants will qualify for modifications. Decisions, however, will be made over the course of the next two years. For those applicants receiving modifications, there will be, as I mentioned earlier, monitoring requirements to assure that their modified discharges are meeting the requirements of Section 301(h).

It will take some time for monitoring and assessing the data received in this program before the final chapter of the debate over secondary treatment and ocean discharges can be written. Only then will we have enough information to assess reliability of programs such as industrial pretreatment programs, toxic control programs, and non-industrial source control programs, which are the substance of Section 301(h).

Our final objective in this effort is to ascertain a level of treatment for ocean discharges which will enable us to achieve the goals and objectives of the Clean Water Act. This may require one level of treatment for POTWs which discharge domestic waste and another level of treatment for POTWs which have industrial waste and toxic materials in their effluent. Once these levels of treatment are ascertained, it may be necessary to revise the Clean Water Act to enable the Administrator of EPA to define ocean discharge requirements or to allow permanent modification of secondary treatment requirements.

As you can see, there still remains much to be done and this will take time. EPA expects that during this period, through continued research efforts by EPA and other U.S. Agencies, and by their counterpart organizations in other countries, such as Japan, and international organizations, we shall come to understand much more about the effects of what we discharge into the oceans. Hopefully, through the continued exchange of ideas, research and technology, we shall be able to eliminate any detrimental effects.

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5/ Secondary Treatment of Municipal Ocean Discharges, United States Environmental Protection Agency, Task Force Report, at 15.

6/ Id, at 2.

7/ The National Commission on Water Quality Staff Report, at IV-76, 77.

8/ The National Commission on Water Quality, Report to the Congress (1976), at 17.

9/ U.S. Senate Debate on the Conference Report, December 15, 1977, 3 Legislative History, at 425.

Improved Operation and Maintenance Opportunities at Municipal Treatment Facilities

John M. Smith*
Francis L. Evans III**
Jon H. Bender***

I. INTRODUCTION

A vital component of the U.S. strategy to meet the National Water Quality Goals of the Federal Water Pollution Control Act of 1972 (PL 92-500) and the 1977 Amendments (PL 95-217) is to substantially improve the performance of publicly owned municipal treatment works through the integration of state and federally-issued permits, federal enforcement actions, and the federal Construction Grants Program (1).

Today there are over 20,000 publicly owned municipal treatment plants in the United States with another 1,000-1,200 new plants being constructed each year. The Federal Water Pollution Control Act of 1972 (PL 92-500) and the 1977 Amendments (PL95-217) established a national municipal effluent standard of secondary treatment for these facilities unless water quality standards require higher levels of treatment.

Although federal and state governments subsidize from 75-98% of the capital costs of publicly owned municipal treatment facilities, the local government bears the responsibility for operation of these plants in accordance with effluent permit requirements established by the National Pollutant Discharge Elimination System (NPDES) mandated by the Federal Water Pollution Control Act (FWPCA) of 1972. The FWPCA of 1972 further provides that municipal facilities failing to meet permit conditions are subject to a range of enforcement actions including fines, administrative orders, and judicial actions for extreme violations. Conformance with permit requirements is based on a self-reporting process for all municipal facilities with supplemental compliance monitoring by state and USEPA Regional personnel.

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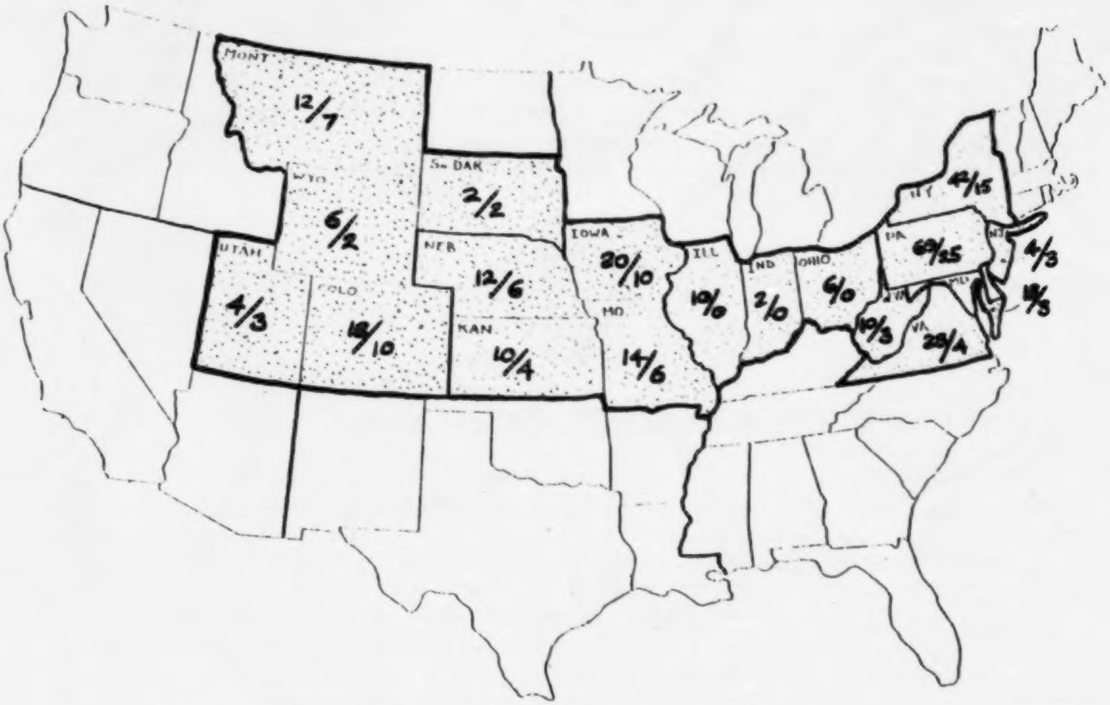
Several national studies since 1975, including legislatively mandated clean water reports to Congress in 1975 and 1976, and a U.S. Government General Accounting Office (GAO) study conducted in 1977 conclusively demonstrated that publicly owned municipal treatment facilities were not meeting permit requirements (2)(3)(4). The GAO study indicated that 40% of all treatment facilities failed to meet design BOD removals and 49% failed to achieve SS design removal efficiencies. The study further documented that only 25% of all trickling filters were in compliance with federal secondary treatment standards for BOD and SS removal. As of 1977, only 50% of all publicly owned municipal treatment plants were in compliance with federally mandated National Pollutant Discharge Elimination System standards.

II. THE NATIONAL SURVEY SUMMARY

Recognizing the significance of the non-compliance problem and the ineffectiveness of the current federal enforcement programs, the USEPA's Office of Research and Development undertook a comprehensive national study of publicly owned municipal treatment plants in 1975 to identify and quantify the specific causes of inadequate performance and to formulate recommendations for improvement. Corollary objectives of the study were to identify future research needs and to demonstrate methods of improved performance. Treatment facilities were selected for study based on sequential screening and selection procedures. USEPA regional offices and state regulatory agencies assisted in initial selection of plants by compiling a list of potential study sites. Plants not meeting one or more of the following general screening criteria were eliminated from the selection process.

1. The plant must incorporate some variation of suspended growth, fixed film, or aerated lagoon biological treatment.
2. The plant should not be severely hydraulically or organically overloaded, nor have obvious identifiable structural or component deficiencies.
3. The plants should range in size up to 38,000 m^3/d (10 mgd) and all major units should be operating.
4. No enforcement action should be underway or pending against the municipality or authority involved.

Site visits were made to a total of 287 facilities to collect more detailed data than the original screening information in order to select those plants at which more comprehensive evaluations would be conducted. The number of site visits, preliminary evaluations, and states included in the survey is shown in Figure 1. These initial site visits required one-half to one full day at each facility to evaluate such things as process configuration; influent and effluent wastewater characteristics; condition of equipment; and discharge permit criteria. Also, the plant superintendent and operating personnel were questioned regarding problems they saw as interfering with plant operations.


MONT
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S. DAK
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UTAH
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COLO
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NEB
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KAN
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IOWA
20/10
MO
14/6
ILL
10/0
IND
2/0
OHIO
6/0
NY
42/15
PA
69/25
NJ
4/3
MD
18/3
WV
10/3
VA
28/4

0/ - indicates No. of site visits per state

/0 - indicates No. of preliminary evaluations per state

Totals 287 site visits
103 preliminary evaluations
18 states

Figure 1. States Including Number of Site Visits and Number of Preliminary Evaluation in USEPA National Survey (5)(6)

In addition to the data collected from the initial 287 site visits, 103 facilities were selected for further comprehensive evaluations. The purpose of the comprehensive evaluations were to examine in detail the system and unit process performance and to evaluate existing operation, maintenance, and administrative practices. Each plant evaluation involved a team of professional engineers and experienced plant operating personnel and required three to five days of onsite field work. In all, 70 potential problem areas were investigated at each facility.

In order to quantify and report the deficiencies and problems at plant sites, both individually and collectively, a plant evaluation summary was developed, consisting of a weighing scale and a ranking table. The scale was devised to rank the 70 different factors that could limit plant performance. For each factor identified at a facility, the extent to which it adversely impacted plant performance was quantified according to the weighing scale points as defined in Table 1. The factors affecting plant performance were then ranked in decreasing order of severity as shown in Table 2.

Table 1

Weighing Scale Used to Quantify Adverse Impact

Weighing Scale	Effect of Specific Factor on Plant Performance
0	No significant effect on plant performance
1	Minor effect on plant performance
2	Minimum indirect effect on plant performance on continuous basis or major direct effect on plant performance on a periodic basis
3	Major effect on plant performance

Based on the results of the comprehensive surveys, the 10 highest ranking causes of poor plant performance result from inadequate plant operation and plant design deficiencies. The highest-ranking factor (#1) was inadequate operator application of concepts and testing to process control. This coupled with the fourth-ranked factor, inadequate understanding of wastewater treatment, indicates that for various reasons operators were not applying the proper concepts of operation to process control. These reasons are attributable to inadequate or incorrect sampling and testing procedures for process control (Factor #2), improper technical guidance (Factor #5), ineffective O&M manual instruction (Factor #9), and significant design deficiencies (Factors #3, 6, 7, 8, and 10), all of which prevent an operator from controlling and "tuning" his treatment system to varying influent hydraulic and pollutant loading characteristics. The 10 leading causes of inadequate performance are described below.

Table 2. Collective Ranking of Factors Limiting Plant Performance

Limiting Factor			No. of Times	
Factor	Area	Description	Factor was ranked #1	Factor was noted
1	Operation	Operator Application of Concepts & Testing to Process Control	24	89
2	Operation	Process Control Testing	0	67
3	Design	Infiltration/Inflow	9	56
4	Operation	Sewage Treatment Understanding	9	50
5	Operation	Technical Guidance	7	47
6	Design	Sludge Wasting & Return Capability	9	43
7	Design	Secondary Process Controllability	3	55
8	Design	Secondary Process Flexibility	3	37
9	Operation	O&M Manual Inadequacy(*)	0	40
10	Design	Aerator	6	27
11	Design	Sludge Treatment(*)	3	36
12	Design	Industrial Loading	4	27
13	Operation	Staff Training	0	31
14	Design	Secondary Clarifier	3	26
15	Operation	Performance Monitoring	0	31
16	Design	Ultimate Sludge Disposal	1	30
17	Administration	Plant Administration, Familiarity with Needs	2	21
18	Design	Disinfection(*)	1	20
19	Administration	Plant Staff - Number	2	22
20	Design	Plant Hydraulic Loading	0	18
21	Administration	Plant Staff - Plant Coverage	0	26
22	Maintenance	Spare Parts Inventory	0	23
23	Design	Laboratory Space & Equipment	0	30
24	Design	Return Process Stream	1	18
25	Operation	Equipment Malfunction	2	17
26	Maintenance	Lack of Preventive Maintenance Program	1	20
27	Design	Alternative Power Source	0	24
28	Design	Organic Loading	7	13
29	Maintenance	General Housekeeping	0	17
30	Maintenance	Maintenance Scheduling & Recording	0	19
31	Administration	Administration Policies	2	15
32	Administration	Plant Staff Productivity	0	17
33	Administration	Insufficient Funding	1	16
34	Maintenance	Manpower	0	14
35	Design	Preliminary Unit Design(*)	0	20
36	Administration	Staff Motivation	0	19
37	Administration	Working Conditions	0	18
38	Design	Alarm Systems	0	19
39	Maintenance	Critical Parts Procurement	0	14
40	Design	Flow Proportioning to Units	0	12
41	Operation	Staff Aptitude	0	13
42	Design	Inoperability Due to Weather	0	12
43	Administration	Staff Supervision	0	13
44	Design	Primary Units(*)	1	9
45	Maintenance	Equipment Age	0	14
46	Operation	O&M Manual - Use by Operators(*)	0	12
47	Administration	Salary	0	12
48	Design	Lack of Standby Units for Key Equipment	0	9
49	Design	Lack of Unit By-Pass	0	12
50	Maintenance	Technical Guidance - Emergencies	0	10
51	Maintenance	Availability of Preventive Maintenance Ref.	0	10
52	Design	Flow Backup	0	7
53	Operation	Staff - Level of Education	0	9
54	Design	Toxic Loading	1	8
55	Design	Submerged Weirs	0	6
56	Design	Plant Location	0	6
57	Operation	Staff Level of Certification	0	8
58	Operation	Staff - Insufficient Time on Job	0	7
59	Maintenance	Staff Expertise - Emergencies	0	9
60	Design	Seasonal Variation Loading	0	7
61	Administration	Unnecessary Expenditures	0	7
62	Design	Process Automation for Control	1	6
63	Administration	Personnel Turnover	0	4
64	Operation	Shift Staff Adequacy	0	3
65	Design	Unit Accessibility	0	3
66	Design	Process Accessibility for Sampling	0	4
67	Design	Process Automation for Monitoring	0	2
68	Design	Equipment Accessibility for Maintenance	0	2
69	Administration	Bond Indebtedness	0	0
70	Design	AWT Units(*)	0	0

*Not included in every plant evaluated

1. **Operator Application of Concepts and Testing to Process Control**—This factor was ranked as the most severe deficiency and leading cause of poor performance at 23 facilities and was a high-ranked factor at a total of 89 out of the 103 plants evaluated. It occurs when a trained operator in a satisfactorily designed plant permits less than optimum performance. This factor was ranked when incorrect control adjustment or incorrect control test interpretation occurred, or when the use of existing inadequate design features continued when seemingly obvious operations alternatives or minor plant modifications could have been implemented to improve performance. The lack of testing and control were not necessarily the result of inadequate training or comprehension in these areas, but simply the lack of or inability to apply learned techniques.

2. **Process Control Testing Procedures**—Inadequate process control testing involves the absence or wrong type of sampling or testing for process monitoring and operational control. This deficiency leads to making inappropriate decisions. Standard unit process tests such as mixed liquor suspended solids, mixed liquor dissolved oxygen, mixed liquor settleable solids, and return sludge suspended solids for activated sludge processes were seldom or never conducted. Also, important operating parameters such as sludge volume index, F:M ratio and mean cell retention time in suspended growth systems or recirculation rates in trickling filter plants were usually not determined. This factor adversely impacted performance at 67 of the 103 plants evaluated.

3. **Infiltration/Inflow**—The results of this widespread problem are manifested by severe fluctuations in flow rates, periods of severe hydraulic overloading, and dilution of the influent wastewater so that both suspended and fixed biological systems are loaded to less than optimal values. The extreme result is the "washout" of suspended growth systems as a result of the loss of solids from the final clarification stage during high flow periods. This factor was ranked first at 56 of the 103 plants evaluated.

4. **Inadequate Understanding of Wastewater Treatment**—This factor is distinguished from Factor #1 in that it is defined as a deficiency in the level of knowledge that individual staffs at various facilities exhibit concerning wastewater treatment fundamentals. On occasion, an operator's primary concern is simply to keep the equipment functional rather than to learn how the equipment relates to the processes and their control. This factor adversely affected performance at 50 plants and was the leading cause of poor performance at nine facilities.

5. **Technical Guidance**—Improper technical guidance includes misinformation from authoritative sources including design engineers, state and federal regulatory agency personnel, equipment suppliers, operator training staff and other plant operators. At any one plant, improper technical guidance was observed to come from more than one source. This factor was ranked as the most severe deficiency at seven plants, and was an adverse factor at 47 facilities.

6. **Sludge Wasting Capability**—This factor was ranked as the leading cause of poor performance at nine facilities and was a factor at 43 plants studied. This factor includes inadequate sludge handling facilities and the inability to measure and control the volume of waste sludge. Either one or both of these conditions was noted as having a major impact on performance at several plants.

7. **Process Controllability**—The lack of controllability was evident in the inability to adequately measure and control flow streams such as return sludge flow and trickling filter recirculation rates. While measurement and control of return activated sludge flow were the most frequent reasons for rating this factor, process controllability was not a major cause of poor performance. It prevented an operator from "tuning" his treatment system to the varying demands which were placed on it by hydraulic and organic loading fluctuations. This factor occurred at 55 plants and was the leading factor at three facilities.

8. **Process Flexibility**—Lack of flexibility refers to the unavailability of valves, piping and other appurtenances required to operate in various modes or to include or exclude existing processes as necessary to optimize performance. Poor flexibility precludes the ability to operate an activated sludge plant in the contact stabilization, step loading or conventional modes and the ability to bypass polishing ponds or other downstream processes to discharge high quality secondary clarifier effluent. Either the lack of, or inadequate, process flexibility was noted as the leading cause of poor performance at three plants and was a factor at 37 facilities.

9. **Ineffective O&M Manual Instruction**—This situation, existing at 40 plants, was judged serious although the adverse effect was moderate. The poor quality of most plants' O&M manuals undoubtedly has contributed to operators' general lack of understanding of the importance of process control and the inability to practice it, but a competent staff could use other available information sources.

10. **Aerator Design**—Deficiencies in aerator design were the major cause of poor performance at six facilities and were less significant factors at an additional 21 plants. Deficiencies were noted in the type, size, shape, capacity, and location of the unit and were of such a nature as to hinder adequate treatment of the waste flow and loading and stable operation.

For each factor shown in Table 2, the area of design, operation, maintenance, or administration is identified. Also shown is the number of times that a factor was ranked Number 1; i.e., the number of times the factor was the leading cause of poor performance, and also the number of plants at which the factor had a "minor" or more serious adverse impact on plant performance. In some cases, plant evaluations did not include every factor being evaluated for potential adverse impact. These factors are marked in the table by an asterisk. However, all factors were noted as having an adverse impact either when the factor was present and a deficiency or an adverse effect was observed or when the factor was not present and an adverse effect resulted from its absence. As noted on the table, operational problems and design deficiencies comprise the top 16 leading causes of poor plant performance.

In addition to the summary information shown here, each of the 103 plant evaluations conducted over a 3 1/2 year period resulted in a report 60 to 100 pages in length, describing in detail the interrelationships of the administrative, operational, design, or maintenance factors affecting performance. The report also documented all operating costs associated with each facility and outlined a specific correction program to optimize performance without major capital improvements. This information is included in EPA's Office of Research and Development national operations and maintenance computerized data base to be discussed later.

Other major conclusions from the National Survey as they affect the potential for improved operations and maintenance of municipal treatment plants are listed below.

1. Lack of aggressive federal enforcement against municipal dischargers has failed to provide needed local government incentive for consistently meeting effluent permit requirements.

2. Fixed film facilities were more often affected by deficient design practices, while suspended growth facilities were more affected by operational problems (5).

3. Total plant staff size, total plant staff cost, specific plant staff size, specific plant staff cost and plant staff salary did not significantly correlate with good or poor plant performance.

4. In nearly all facilities surveyed, adequate manpower was provided for proper plant operations and maintenance. Plant maintenance was satisfactory, but plant operations was unsatisfactory even though a greater proportion of the operator's time was spent conducting "operations" tasks.

5. Current operator practices for the smallest facilities surveyed, 0-380 m^3/d (0-0.1 mgd), were poor. For larger facilities surveyed, 3800-38,000 m^3/d (1.0-10.0 mgd), operator practices were only fair. (These conclusions based on 30 plants)(5).

6. For all of the facilities studied, operation and maintenance costs varied from \$0.25/m^3 (\$0.93/1000 gal) for 380 m^3/d (0.1 mgd) facilities to \$0.03/m^3 (\$0.10/1000 gal) for 76,000 m^3/d (20 mgd) facilities.

7. In a sub-set of 30 plants studied, significant differences in O&M costs between suspended growth and fixed film facilities were noted as shown below.

Type Plant	Plant Size m^3/d (mgd) 0.0-380 (0.0-0.1)	 380-3,800 (0.1-1.0)	 3,800-38,000 (1.0-10.0)
Suspended Growth			
O&M Cost \$/m^3	0.25	0.12	0.06
(\$/1000 gal)	(0.93)	(0.45)	(0.21)
Fixed Film			
O&M Cost \$/m^3	--	0.09	0.03
(\$/1000 gal)	(--)	(0.35)	(0.10)

8. Although maintenance factors were cited in some instances as contributory to inadequate performance, they were not a significant or dominating factor in most of the plants studied.

9. O&M manuals were noted to be of marginal value during the study.

10. On-site training including "trouble-shooting techniques" to increase operators' understanding of process control was strongly recommended as an effective remedial solution.

11. Plant staffing and staff salaries, although less than that recommended by USEPA staffing guidelines in some instances, were not considered major factors.

12. For 30 of the plants studied, average operation staff salary ranged from \$10,501/year for a 0.0 to 380 m^3/d (0.0 to 0.1 mgd) facility size range to \$13,107/year for a 3,800 to 38,000 m^3/d (1.0 to 10.0 mgd) size range. Total salaries for these 30 plants ranged from 17.2% to 66.3% of total plant operating costs with an average of 46%.

III. A NEW APPROACH FOR IMPROVING PERFORMANCE

In a critical evaluation of the data from the above National survey, it was noted that at each treatment facility a combination of factors limiting performance was always observed and that a single cause of poor performance at any one facility was never observed. Because there is an interrelationship between performance limiting factors and corrective programs, and because most existing correction programs focus on single problems only, a new approach which addresses all problems at a single facility was developed as more effective in improving existing plant performance. This approach is called a Composite Correction Program (CCP). The purpose of the CCP is to eliminate all the performance limiting factors at a plant through the implementation of the correction recommendations that are made as a part of the comprehensive evaluation. The general approach is to improve the process control decision making capability of the plant superintendent and the operators and to eliminate administrative, maintenance, and operations factors which were preventing optimum performance of the treatment system. In a few cases, minor plant modifications were recommended that could be made either by the plant staff or a local contractor.

Among the services provided during the on-site technical assistance were the establishment of process control monitoring and testing programs, operator training to improve the application of concepts and testing to process control, adjustment of plant staffing and rescheduling of staff activities, minor modification of a clarifier, and modification of plant operations to optimize and maintain the performance of the biological units.

The CCP was successfully demonstrated at several facilities. When the program was implemented at the Havre, Montana Wastewater Treatment Plant, a significant improvement in plant effluent quality resulted and permit standards could be met consistently. At the Havre plant, the effluent quality for six months prior to implementation of the CCP averaged 31 mg/L for BOD and 30 mg/L for TSS. Both BOD and TSS concentrations averaged less than 10 mg/L for an eight-month period following initiation of the CCP and development of desired activated sludge characteristics. During that period of time, the plant's BOD loading increased by 27%, yet BOD discharged to the receiving stream decreased by 68%.

At other facilities where the CCP technical assistance approach was used, improved performance resulted from changes in plant operations or minor changes in plant design features. The improvement in effluent quality that was achieved at 12 of the plants evaluated is shown in Table 3.

The significance and impact of a CCP approach to optimizing plant performance are indicated by improved effluent quality at the demonstration facilities and by the potential improvements which could be realized if such a program were implemented at all the facilities at which comprehensive evaluations were performed.

Of the 103 facilities evaluated, only 37 plants (36%) were meeting their respective NPDES standards consistently. However, if as a result of the evaluations, the recommendations were implemented, it was estimated that an additional 51 treatment plants could consistently meet NPDES standards, and 88 plants (86%) would achieve optimal levels of performance beyond which further improvement in effluent quality would not be possible without upgrading the existing facilities.

Table 3. Plant Performance Improvement Thru Implementing CCP

FACILITY	ACTUAL FLOW m^3/d (MGD)	EFFLUENT QUALITY				PLANT TYPE - COMMENTS
		BOD_5 (mg/L)		TSS (mg/L)		
		BEFORE	AFTER	BEFORE	AFTER	
022	45 (0.012)	35	10	60	10	Activated sludge, extended aeration, with polishing pond. The return sludge flow rate was excessively high, causing solids loss over the clarifier weir. The R/Q ratio was reduced from about 1000% to about 100%, and excessive solids loss was eliminated. A pond bypass should be installed to discharge good final clarifier effluent to the receiving stream.
029	5,190 (1.37)	31	9.7	30	10	Activated sludge, conventional, without primary clarifier. Changes in return and waste sludge control procedure were implemented and a plant deficiency was corrected.
048	1,300 (0.34)	68	10	116	10	Activated sludge, conventional, without primary clarifier. Insufficient sludge wasting and sludge bulking. Increased wasting controlled sludge bulking. An I/I problem should be corrected.
050	640 (0.17)	45	10	80	10	Activated sludge, extended aeration, without primary clarifiers. Insufficient sludge wasting was corrected. Increased wasting controlled sludge bulking. Operations testing and process controls were implemented.
053	420 (0.11)	32	10	50	10	Activated sludge, extended aeration, without primary clarifiers. Insufficient sludge wasting was corrected. Decreased return, controlled scum withdrawal, and increased wasting, was well as implementation of operations testing and process controls were completed.
060	1,900 (0.49)	45	35	37	27	Two-stage trickling filter with primary (second-stage is an activated bio-filter system). A valve leaking mixed liquor to the chlorine contact basin influent was found. Plant effluent is still marginal because of design limitations in aeration capacity.
061	640 (0.17)	37	10	42	10	Activated sludge, contact-stabilization, without primary clarifier. Inadequate sludge wasting and return control provided. Return flow rate not controlled. Operations testing and process control were implemented and good effluent quality maintained. A piping modification should be completed to operate conventionally.

Table 3. (Continued)

FACILITY	ACTUAL FLOW m^3/d (MGD)	EFFLUENT QUALITY BOD_5 (mg/L)		EFFLUENT QUALITY TSS (mg/L)		PLANT TYPE - COMMENTS
		BEFORE	AFTER	BEFORE	AFTER	
065	490 (0.13)	72	21	143	8	Activated sludge, contact stabilization without primary clarifiers. Solids loss occurred due to inadequate sludge mass and return sludge control. Minor modifications were made to the return sludge and wasting mechanism and to the chlorine contact tank.
082	310 (0.083)	50	20	75	30	Activated sludge, contact stabilization and parallel trickling filter. Flow splitting to the two plants was not optimized. Solids loss from the contact stabilization plant occurred repeatedly due to inadequate process control. The digester was used as part of the reaeration basin. Sludge beds must be subsidized with wethaul of sludge for ultimate disposal.
085	3,200 (0.84)	50	10	108	10	Activated sluge, oxidation ditch without primary clarifiers and with sludge lagoons. Excessive solids loss due to inadequate process control understanding. Mass control and process control testing were improved. Ultimate sludge disposal capability must be expanded.
086	1,800 (0.48)	75	8	150	5	Activated sludge, extended aeration without primary clarifiers and with sludge lagoons. No wasting was practiced. Existing sludge lagoons have inadequate capacity. A sludge truck for land application of sludge was purchased.
097	3,200 (0.84)	23	10	34	15	Activated sludge contact stabilization without primary clarifiers, with aerobic digester and roller press sludge dewatering. The approach to process control was misdirected and was leading towards poor performance. Process control understanding was increased and impending problems were avoided. I/I problems anticipated in isolated cases.

Because of the demonstrated success of the CCP developed in this survey, the USEPA's Office of Enforcement has adopted the procedure for widespread use as a part of its national enforcement strategy for publicly owned municipal treatment facilities.

To assist in this effort, USEPA's Office of Research and Development has expanded and refined the plant evaluation and CCP procedure into a protocol that is applicable to a wide range of plant sizes and types and contains the necessary cost estimating guidelines, evaluation checklists, and other necessary information to be used by consulting engineers throughout the country for improving plant performance.

Recent statements from Eckardt C. Beck, Assistant Administrator for USEPA's Water and Waste Management Program, and Jeffrey G. Miller, Acting Director for USEPA's Enforcement Division, excerpted from an October 10, 1979 policy address at the Water Pollution Control Federation annual meeting in Houston, Texas indicate EPA's increasing concern for improving the operation and maintenance of the Nation's municipal treatment facilities as well as the use of previously-mentioned protocols. The text of this statement is shown below.

Sewage Treatment

EPA LAUNCHING NEW DRIVE AGAINST IMPROPER PLANT OPERATION

HOUSTON, Tex. — (By an Environment Reporter Staff Correspondent) — Proper operation and maintenance of municipal sewage treatment systems is now a top concern at the Environmental Protection Agency, and two key agency officials here said treatment system management will be emphasized in EPA's water and enforcement programs.

Eckardt C. Beck, Assistant Administrator-designate for EPA's Water and Waste Management programs, October 10 told a meeting of the Water Pollution Control Federation improper municipal treatment system operation and maintenance is "one of the major reasons" why almost 60 percent of the municipalities nationally failed to meet the 1977 Clean Water Act deadline for compliance with secondary treatment standards.

Beck said "O&M" now stands for "Operation and Management" in the municipal sector. Management, he added, "means more than polishing the knobs and pipes."

Beck told WPCF conference attendees the benefits of EPA's construction grant program — plant expansion and improved treatment processes — will be negated without proper plant management. In some instances management alone will result in a plant complying with secondary treatment requirements. "If you can achieve compliance without major new construction — do it," he said.

Use of Contractors Urged

Beck said in some instances the key to proper plant management may be for a municipality to contract with a private firm for plant operation and maintenance.

"I am a firm believer that the solutions to our environmental problems demand an involvement of the private sector," Beck said. He also announced EPA is now using demonstration grants "to act as a catalyst to attract more private firms into the business of upgrading plant services."

Jeffrey G. Miller, acting director for EPA's enforcement division, told the WPCF conference attendees the "tremendous disparity" between municipal and industrial compliance with Water Act requirements has led to a "shift of emphasis from industrial to municipal" dischargers.

Miller said improper municipal treatment plant operation is a major reason why so many systems failed to meet the Water Act's 1977 secondary treatment deadline, and he promised increases in the number of enforcement actions.

Miller said municipalities do not "feel in a position of risk" and therefore have little incentive to operate treatment systems efficiently. "It's plain we have to change that attitude," he said, and he urged listeners to look for a "trend" as treatment plant supervisors are sued, and court appointed experts take over management of municipal systems.

"The message must get out" that cities are at risk, Miller said. He predicted EPA and Justice will bring about 50 to 60 cases against municipalities that fail to properly operate their treatment systems.

Miller said that in many of the cases Section 311 of the Water Act will be used to require "immediate clean-up" of municipal discharges. He also said EPA is developing a diagnostic protocol that can tell plant operators why their system is not meeting discharge standards.

Miller said the test procedure will cost about $15,000 and may in the future be tied to National Pollutant Discharge Elimination System permit issuance or resolution of enforcement actions.

In addition to the national interest in adopting the plant evaluation-CCP approach, the State of Colorado was recently awarded an EPA research cooperative agreement to demonstrate the cost effectiveness of the approach for 20 municipal treatment plants as a part of the state's enforcement activities. Another state in EPA's Region 5 is also planning a state wide demonstration of the CCP.

It is estimated that over 70% of the non-complying plants in the U.S. can be brought into compliance with NPDES permits using this approach at a plant cost of $10,000 to $25,000 (1977 dollars) per plant depending on plant size and complexity (5)(6).

IV. IMPACT OF NATIONAL SURVEY IN DIRECTING ORD RESEARCH INCLUDING RECENT RESULTS

In addition to providing an in-depth understanding of the four major categories of problems affecting the performance of municipal treatment plants and the development of the CCP, the National Survey results have been the major influence in directing USEPA Office of Research and Development programs in plant operation and design in fiscal years 78 through 83.

This program, operated on a national basis under the authority of Title I of the Federal Water Pollution Control Act, has the following goals:

- Reduce the construction and operating cost of municipal collection and treatment systems
- Improve municipal treatment plant performance and reliability
- Reduce energy consumption
- Provide baseline cost and performance data to support operating program management and administration and regulatory activities
- Support Agency enforcement activities

The research program is divided into four specific areas to accomplish the above goals. The specific areas of research, research projects, and National Survey problem areas addressed are shown in Table 4.

While it is not possible to discuss each of USEPA's current research projects, recent research activity and Agency emphasis in the areas of: (a) identifying specific design deficiencies; (b) development of design guidelines; (c) improving plant reliability, and (d) improving plant operations and management will be discussed in the following sections.

Identification and Proposed Methods for Reducing Design Deficiencies in Municipal Treatment Plants

Design deficiencies in municipal treatment plants is a very broad classification of problems that were found to be present in varying degrees in nearly every plant studied in the National Survey. Because of the severity and widespread distribution of design deficiencies, a separate research program was undertaken in 1978 by EPA's Office of Research and Development to identify and analyze the problems and to develop a design deficiency matrix and correction manual to be used by consulting engineers during the planning and design of new facilities, and for state and EPA authorities for their use in reviewing and approving plans and specifications for the construction of federally funded facilities (7).

The manual has also been designed for use in correcting design deficiencies at existing municipal plants. The manual describes design deficiencies that can increase plant operation and maintenance requirements, annual operating costs, energy requirements, and can contribute to performance and reliability problems, poor safety practices, and/or decrease the flexibility of plant process control. The manual is intended to provide design engineers with guidance that will make their designs more operable and maintainable at less cost as well as more flexible in providing adequate performance during times of changing influent characteristics. The manual is divided into three main sections.

Table 4. USEPA Current Plant Operation and Design Research Program (1980)

RESEARCH PROGRAM	NATIONAL SURVEY PROBLEM AREAS			
	A	D	O	M
DESIGN AND ENGINEERING AND COST-EFFECTIVENESS ANALYSIS				
Develop a Decision Optimization Methodology for Residuals Management		●		
Mgm't Information System for Toxics-C/E of Pretreatment; NPDES Impact	●		●	
Energy Assessment Procedures Manual		●	●	
O&M Practices of Land Treatment Processes		●		
* Impact of Recycle and Return Flows on Plant Performance			●	
* Low Cost Correction of Design Deficiencies		●		
* Impact of Peak Flows on Conventional Process Performance		●	●	
* RBC's Performance, Operation & Design		●	●	
* Computerized Data Base-O&M Cost, Design & Performance	●	●	●	●
Integrated Community Utility Systems		●		●
Operation and Performance of Fine Bubble Aerators		●	●	
Improving the Basis for Design		●		
* Design Information Series		●		
* Analysis of O&M Costs of Conventional Collection Systems		●		●
Dev. & Implement a Cost Analysis System for WW Collection Systems	●		●	●
Demonstrate O&M Cost Reduction in WW Collection & Treatment Systems	●		●	●
IMPROVEMENT IN SYSTEMS RELIABILITY				
* Identify & Quantify Factors Affecting Reliability; incl. Toxics		●	●	
* Mechanical Reliability of Conventional Components of Plants		●	●	
* Improving Clarifier Performance thru Improved Hydraulics		●	●	
Demonstrate Improved Clarifier Performance		●	●	
Develop Methodology & Guidelines for Systems Reliability Analysis	●	●	●	●
Evaluate Private Sector Expertise to Conduct CCPs	●		●	
Survey & Performance Analysis of POTW Contract Operation	●		●	
Methodology for Evaluating Alternative Water Quality Management	●			
Analysis of Methods for Evaluating Established Stnds. & Monitoring Requirements for Water Quality Monitoring	●			
Evaluate Feasibility & Demonstrate Fail-Safe Treatment Processes	●	●	●	●
IMPROVEMENT IN PLANT DESIGN AND OPERATION				
* Areawide Demonstration of CCP	●		●	●
Areawide Demonstration of CCP - 2nd Site	●		●	●
Develop & Evaluate Effectiveness of On-Site Training Programs	●		●	
Optimize Opertion of a Group of Small WWTPs	●		●	
Evaluate & Demonstrate Centralized Management of O&M Activities	●		●	●
Computer Program Development for Centralized Management	●		●	●
Computer Program Dev.-C/E of Alternative Systems in Urban Areas		●		
Protocol for Comprehensive Plant Evaluation	●		●	●
Improving O&M and Design of Land Application Systems	●	●	●	●
Engineering Eval.and Analysis-Extramural Support for In-House Studies	●	●	●	●
AUTOMATION AND INSTRUMENTATION				
Develop & Demo. Control Strategies-Automation of Biological WWTPs	●		●	●
Assessment of Control Strategies-Eval.Field Data,Computer Simul.(T&E)			●	
Eval. Intelligent Computer to Monitor & Control Operations of WWTP		●	●	
Feasibility Study for Establishing an Instrument Certification Lab	●		●	●
Dev. Specs & Testing Protocol for On-Line Measuring Inst. & Auto.	●		●	●
Dev. Organ. Design & Management Plan for Inst.Cert.Lab.	●			
Support of Instrument Certification Lab	●			
Evaluate Application of Computer Technology to Control of WWTPS	●		●	

* - Major emphasis on these programs in fiscal years 1979, 1980 and 1981

A - Administration
D - Design
O - Operation
M - Maintenance

- Design Deficiencies Matrix
- Design Consideration Modules for New POTWs
- Design Deficiency Correction Modules for Existing POTWs

Design Deficiencies Matrix

Design deficiencies commonly found in POTWs are categorized in a matrix format. The 14 categories used and the number of deficiencies addressed in each category are listed below.

Number	Category	No. of Deficiencies
1.0	General Plant Design	884
2.0	Preliminary Treatment	226
3.0	Primary Treatment	82
4.0	Air Activated Sludge	392
5.0	Oxygen Activated Sludge	18
6.0	Trickling Filter	110
7.0	Disinfection	76
8.0	Anaerobic Digestion	134
9.0	Aerobic Digestion	62
10.0	Sludge Dewatering	318
11.0	Lagoons	162
12.0	Land Application	574
13.0	Sludge Disposal	66
14.0	Sludge Reduction	156

Within each of the above categories the deficiencies are further grouped according to type and then numerically referenced to unit operations and components specific to the category. The deficiency groups used include:

- Layout, arrangement, and placement of components in design of plant
- Hydraulic design considerations
- Mechanical design considerations
- Electrical/instrumentation design considerations
- Safety considerations
- Environmental considerations

Design Consideration Modules for New POTWs

The design deficiencies listed in the matrix are discussed in "Design Consideration Modules," in terms of items that should be reviewed by the engineer during design of a new or expansion of an existing wastewater treatment plant. Features of the module format are as follows:

- The report section, deficiency matrix category, and applicable unit operation or component are indicated in the upper left-hand corner. This information allows the reader to cross-reference the deficiency matrix with the design consideration modules.

- A treatment plant block flow diagram is presented in the upper right-hand corner. The deficiency category represented by the module is shaded to allow the reader to visually identify the portion of the plant discussed.

- The design deficiencies, including reference number and description, listed in the matrix are presented in a column on the left side of the page and the associated design consideration discussed in a column opposite the deficiency.

- The module page number also identifies the deficiency category discussed. The first, second, and third digit of the page number represent the deficiency category, unit operation/component, and actual page sequence, respectively.

Design Deficiency Correction Modules for Existing POTWs

Suggested methods to correct the design deficiencies presented in the matrix, when already present at an existing wastewater treatment facility, are described in "Design Deficiency Correction Modules." The format of these modules is the same as the "Design Consideration Modules." In addition, where applicable, the correction procedure includes required relevant information such as methods, materials, cost, and a sketch.

The manual is used by locating the design deficiencies in the matrix organized by category and deficiency group, then by referring to the appropriate design consideration modules for new plants, or to the design deficiency correction module for existing plants.

The manual includes 3,260 identified deficiencies with correction modules and design consideration modules for each. The manual will be published by USEPA in mid-1980 and distributed to all consulting engineers and state and federal review authorities.

It is anticipated that the use of this document will have a significant impact in reducing the occurrence of design deficiencies in new plant construction. Its impact in correcting deficiencies in existing plants will depend on increased incentives for improved operation through more aggressive state and EPA enforcement activities.

Development of Design Guidelines

Because of USEPA's role as a regulatory agency, it has been concerned with the establishment of standards for, and regulation of pollutant discharges to the environment rather than being actively involved in specifying, controlling, or significantly impacting the design of facilities for meeting necessary levels of control. Design practice, and changes in design practice, have been left to professional societies such as the American Society of Professional Engineers, the Water Pollution Control Federation, the Ten State Standards Committee, Consulting Engineers Council, and the Boards of Health of the various states, and other scientific and professional groups.

This role has changed somewhat in recent years for municipal treatment plants due to the large increase in federal funding of these facilities, the overall increase in both capital and operating costs, and the recognition of the impact of deficient designs and inappropriate design criteria on these costs.

In an attempt to improve current design practice without being prescriptive, EPA's Office of Water Program Operations and Office of Research and Development have initiated a new series of design information and guideline documents (DIGS) to be prepared by EPA in concert with leading consultants and experts in the field. These information and guideline documents will be based on the latest results from EPA sponsored and other full scale research studies, full scale performance data and other state-of-the-art technical information and will provide in a condensed format, a synopsis of both the methods and scientific basis for the rational design of municipal treatment works unit processes, systems, and system components. Improved guidance and design information will be disseminated for both conventional and newer technologies through the publication of four to six documents per year for the next five years. The documents will undergo an intensive two-step peer review process to insure the highest possible technical quality before final publication. The overall development of the information and guideline documents is under the direction of an executive committee including membership from EPA, ASCE, and WPCF. The first five documents, to be finished in 1981, are for the subjects of: (a) hydraulic peaking; (b) rotating biological contactor design; (c) land application; (d) effects of sidestreams on biological treatment plants; and (e) aeration devices.

Improving the Reliability of Municipal Treatment Facilities

USEPA research activities for improving the reliability of municipal wastewater treatment plants were initiated in January 1978, and are scheduled for completion in 1981.

The program evolved because: (a) current USEPA reliability guidelines for municipal treatment plants were outdated; (b) the complexity of new secondary treatment facilities had sharply increased; (c) a much greater choice of mechanical components and unit process variations was being incorporated into new designs; and (d) USEPA has become increasingly concerned about the effect of toxic materials on biological processes and the ability of these processes to remove these materials.

The approach chosen to investigate overall plant reliability was to classify the nearly 120 separate unit processes or operations currently used for treatment into the three categories of: (1) biological treatment, (2) physical and chemical processes, and (3) mechanical components. The next step was to investigate each of these classifications in detail to form the necessary data base for conducting a systems reliability analysis. The ultimate goal is to describe the least costly system design configuration for achieving a given effluent quality reliability.

Biological Stability/Reliability

Research studies to define the stability and reliability of both activated sludge and trickling filter plants were conducted for USEPA's Office of Research and Development by the University of California at Davis, California. The results of these studies as described in draft final reports and a paper submitted for publication in the WPCF Journal are summarized below (8)(9)(10).

Effluent Standards

Before discussing stability and reliability relationships for biological treatment plants, it is necessary to review the effluent standards for secondary treatment that form the design criteria for 64% of the new facilities that will be constructed in the U.S. between now and the year 2000 (11). The effluent limitations for secondary treatment defined by USEPA under PL 92-500 are as follows:

> "The arithmetic means of the secondary treatment effluent BOD and SS concentrations shall not exceed 30 mg/L in a period of 30 consecutive days, nor shall exceed 45 mg/L in a period of 7 consecutive days. In addition, the arithmetic means of the concentrations of BOD and SS remaining in the effluent over any 30-day period shall not exceed 15 percent of the arithmetic mean of the values in the influent (85 percent removal)."

It is important to note that this standard is based on an arithmetic mean for BOD and SS concentrations and that the standards do not recognize the natural variability of biological plant performance, nor do they readily permit the use of probabilistic design methodologies.

Study Approach

The study approach was to analyze statistically the performance of 43 activated sludge plants and 11 trickling filter plants to examine the variability of effluent quality and to examine the effects of influent characteristics, operational factors, design criteria, and reactor configurations on plant stability and reliability. Initial correlation and regression analysis for the group of activated sludge plants showed that consistent relationships between effluent concentrations, influent loading, biological and operational variables did not exist for the group. Better correlation was found possible for single plants, particularly for plants not performing properly (8).

The analysis of plant effluent variability for BOD and SS for both the activated sludge and trickling filter plants clearly defined the distribution of effluent values and led to the formulation and verification of a potentially powerful probabilistic design model for both processes. Only the study results pertaining to activated sludge will be summarized in this paper.

Conclusions of Stability/Reliability Analysis

The following conclusions were reached based on a detailed study of 43 activated sludge plants ranging from 2,300 to 791,000 m^3/d (0.6 to 209 mgd) (8).

Performance

1. Effluent concentrations of BOD and SS varied significantly among the plants in this study. Mean effluent BOD values ranged from 1.9 to 85.6 mg/L with a group average of 15.8 mg/L. Mean values for effluent SS ranged from 2.5 to 80.1 mg/L with a group average of 19.4 mg/L. On the average, the effluent SS concentration and its variation is greater than the corresponding effluent BOD values.

2. Solids separation facilities are presently a limiting factor in the performance of activated sludge treatment systems.

3. Effluent concentration distributions are not symmetrical and generally are skewed farther to the right than to the left of the most frequent value.

4. A lognormal distribution was found to fit consistently the observed effluent BOD and SS data best.

Reliability

5. A probabilistic approach to process design provides a theoretical basis for the analysis of process reliability.

6. The Coefficient of Reliability (COR), defined on a probability basis, can be used to relate mean constituent values to selected discharge standards.

7. A reliability model has been developed that can be used for both the estimation of the mean value in design of a process or for the prediction of effluent quality in a plant under operation.

Stability

8. "Standard deviation" is the most appropriate measure of stability.

9. Plants having a standard deviation of less than 10 mg/L for both effluent BOD and SS may be considered as "stable" statistically; plants with a standard deviation of greater than 10 mg/L may be considered "unstable".

10. The mean effluent BOD for stable plants is usually less than 17 mg/L, the mean effluent SS values is less than 15 mg/L. Effluent BOD and SS concentrations in stable plants rarely exceed 60 mg/L.

11. Unstable plants have annual mean effluent BOD and SS values greater than 16 mg/L and 10 mg/L, respectively. The maximum daily BOD and SS concentrations of these plants are usually much greater than stable plants and exceed 90 mg/L and 130 mg/L, respectively.

12. Process upsets occur more often due to high values of SS as opposed to high values of BOD in the final effluent.

13. There is no relationship between the size of plant and stability.

14. Various activated sludge processes ranked in terms of decreasing stability and reliability are as follows: (1) step-feed and step aeration; (2) conventional; (3) complete-mix, and (4) contact-stabilization.

Variability

15. Generally, no single variable can be used to characterize the variability of effluent BOD and SS in all plants. Coefficients of correlation between effluent concentration and other variables are insignificant and vary in both magnitude and sign.

16. No significant time-related trends or cycles were found for activated sludge effluent data.

17. The effect of the lag period between influent and effluent variables is insignificant.

18. The combination of all available variables (input, environmental, and operational) can only be used to explain between 13% and 61% of the effluent BOD and between 5% and 52% of the effluent SS fluctuations (depending on the plant). The remaining variations (due to inherent or human factors) could not be explained in the study.

19. Variables controlling plant performance vary from plant to plant. In most cases, one or two variables can be used to explain more than 90% of the total variability that was accountable.

20. Among all variables, wastewater temperature was the most significant in accounting for effluent variations. Other variables found to be significant (minimum significance of 5%) depending on the plant include time, flow, SVI, influent BOD, MCRT, MLSS, and influent SS.

21. In many plants, the mean cell residence time (MCRT) was not properly used for plant operation. Only a few plants were operated in such a way that the MCRT was a useful operating parameter.

Model Formulation

The reliability model developed during the study relates the mean constituent value m_X (i.e., the design value), to a standard, X_s (as set by USEPA), that must be achieved on a probability basis. The model will be summarized here. The reader is referred to the original work for details of the development (8)(9)(10)(12).

For log normal distribution of effluent BOD and SS values from activated sludge plants, the following expression can be derived for the Coefficient of Reliability (COR) (8).

$$\text{COR} = \frac{m_X}{X_s} = (V_X^2 + 1)^{1/2} \exp\left\{-Z_{1-\alpha}[\text{Ln}\,(V_X^2 + 1)]^{1/2}\right\} \qquad (1)$$

where m_X = mean constituent value (design value)
X_s = a fixed standard
V_X = coefficient of variation
$Z_{1-\alpha}$ = a number with the property that $P(Z \leq Z_{1-\alpha}) = 1 - \alpha$ where Z is a normally distributed variable with mean zero and variance P one. The value of $Z_{1-\alpha}$ may be obtained from standard normal variate tables.
Ln = natural logarithm

Design Procedures

The above equation can be used to design a wastewater treatment process to meet an effluent quality equal to X_s a stated $1-\alpha$ percent of the time when the coefficient of variation V_X for the particular process is known. Analysis of effluent quality from the 43 activated sludge facilities in this study resulted in the Coefficients of Variation shown in Table 5.

Table 5. Statistics of the Annual Effluent BOD & SS Concentration Data for Different Process Types (8).

Process type	No. of plants	Mean		Standard deviation		Coef. of variation	
		$\bar{X}$	S_x	$\bar{X}$	S_x	$\bar{X}$	S_x
		BOD					
Conventional	18	12.80	6.85	9.54	7.99	0.69	0.25
Complete-mix	5	16.82	6.67	13.24	6.51	0.77	0.13
Step-Feed/Aeration	13	10.84	7.68	8.28	7.56	0.68	0.25
Contact Stabilization	4	38.38	32.08	28.17	20.92	0.76	0.14
Extended Aeration	1	14.41	—	5.31	—	0.37	--
Kraus	1	24.02	—	9.85	—	0.41	--
Aerated Lagoon	1	30.35	—	11.61	—	0.38	--
All Plants	43	15.76	13.43	11.28	10.49	0.70	0.23
		SS					
Conventional	18	14.92	10.53	16.02	18.61	0.86	0.38
Complete-Mix	5	19.88	14.19	19.65	16.92	1.00	0.51
Step-Feed/Aeration	13	16.23	16.65	16.83	23.89	0.83	0.34
Contact-Stabilization	4	40.88	26.75	37.66	26.18	0.90	0.14
Extended-Aeration	1	8.82	--	5.28	--	0.60	--
Kraus	1	24.12	--	9.26	—	0.38	--
Aerated Lagoon	1	58.79	—	18.71	--	0.32	-
All Plants	43	19.40	17.03	18.35	20.60	0.84	0.37

After selection of X_S and $1 - \alpha$, the COR for the process is calculated according to Equation (1) and then the mean constituent value (m_X) necessary to meet the X_S standard is computed. The mean constituent value is then used for kinetic modeling and other process design considerations. A most significant advantage of using this approach is being able to describe the coefficient of variations of different biological processes, and by using the (COR) to design to meet probabilistic rather than deterministic effluent standards.

During this model development, the stability of the activated sludge process was studied. Examination of the descriptive statistics for the 43 facilities resulted in the conclusion that plants having a standard deviation of less than 10 mg/L for both effluent BOD and SS were considered stable. The mean constituent value (m_X) for this case would be equal to the standard deviation divided by the process coefficient of variation.

A related finding of this study as it applies to the relationship between activated sludge plant design and effluent standard setting is that deterministic standards should be based on the geometric mean rather than the arithmetic mean for parameters that are lognormally distributed.

Justification of this approach is predicated on the concept that the distribution parameters (mean and standard deviations) are simply related to the distribution function and more accurately describe the central tendency of the data. Geometric mean data lends itself very well to analysis of multiple time periods such as 7 and 30 day averages since the mean and standard deviation of geometric mean values for any time period are easily calculated.

Physical Process Reliability

As noted from the above study, physical process reliability is an integral part of activated sludge plant design and has a significant influence on the coefficient of variation. In particular, the variability of SS is noted to be greater than the variability of BOD for activated sludge plants and solids separation facilities (final clarifiers) were noted to limit the performance of activated sludge plants. Because of the large potential for improved secondary clarifiers to increase overall activated sludge treatment efficiency, EPA's Plant Operation and Design Research Program has initiated several studies examining the hydraulic performance of secondary clarifiers as the first in a series of studies of the reliability and performance of physical and chemical processes.

One of the major differences in the design of secondary clarifiers and other process design decisions that affect overall plant performance is that the geometry of both rectangular and circular clarifiers are prescribed, to a large extent, by the standard designs of equipment manufacturers. Most secondary clarifiers are sized based on gross hydraulic loading in terms of average and peak overflow rates and little consideration is given to differences in settleability of mixed liquor as it affects the shape and size of stilling basin design, flow distribution within the clarifier, development of surface area of the clarifier, or the effects of sludge blanket height and rates of sludge withdrawal.

The initial physical process reliability study was conducted by Crosby, Young, and Associates under contract to USEPA (13). This project included the study of eight full-scale clarifiers where performance problems were thought to exist. The overall approach was to monitor clarifier baseline performance, conduct dye tracer studies to identify hydraulic characteristics, modify clarifier inlet baffling or stilling basin design, and to document the changes in performance due to the modifications. The location and physical characteristics of the clarifiers studied are shown in Table 6. Both rectangular and circular final clarifiers from both air and oxygen activated sludge plants were included in this study.

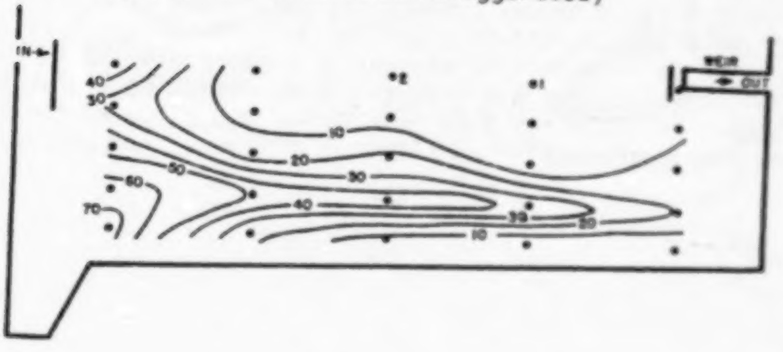
Table 6. Clarifier Locations and Physical Characteristics (13)

Location	Process Type	Design ($m^3/d \times 10^3$)	Clarifier Type	Dimensions (m)	Depth (m)	Design Surface Overflow Rate ($m^3/m^2/d$)	Sludge Removal Mechanism
Albuquerque, NM	Air Activated Sludge	148.00	CFPO	41 (diam)	4.0	27.8	Siphon
Dallas, TX - Ten Mile Creek Regional Waste Treatment Plant	Air Activated Sludge	25.50	CFPO	32 (diam)	3.7	32.1	Siphon
Holly Hill, FL	Mechanical Activated Sludge	4.54	R	20 (L) 4.9 (W)	2.7	23.5	Chain & Flight
Morganton, NC	Pure O_2 Activated Sludge	30.30	CFPO	24 (diam)	4.9	32.5	Siphon
Oakland, CA - East Bay Municipal Utility District							
Main Plant	Pure O_2 Activated Sludge	636.00	PFPO	43 (diam)	4.3	26.5	Siphon
Process Water	Air Activated Sludge	3.78	CFPO	14 (diam)	3.0	25.6	Siphon
Orange County, FL Sand Lake Wastewater Treatment Plant	Air Activated Sludge	114.00	R	63 (L) 23 (W)	4.0	19.7	Traveling Bridge Siphon
Stamford, CT	Mechanical Activated Sludge	75.70	CFPO	40 (diam)	4.0	30.6	Siphon

CFPO - center feed, peripheral overflow
PFPO - peripheral feed, peripheral overflow
R - rectangular

The analytical approach to evaluate each clarifier was to continuously inject Rhodamine dye into a clarifier inlet stream and to monitor the dye concentrations at 25 predetermined points in the clarifier at 30-minute intervals for a period of 1 1/2 to 2 hours. Suspended solids were also measured at the 25 sampling points to permit correlation with dye concentrations. Dye dispersion and solids wave tests were also conducted to indicate directional preferences in the clarifier and to determine the effect of sludge removal mechanisms on clarifier hydraulics. Typical flow patterns for one such test are shown in Figure 2.

Figure 2. Typical Results of Flow Pattern Test Showing Dye Concentrations in a Longitudinal Section of Clarifier After 15 Minutes of Continuous Dye Release (Note: Vertical Scale Exaggerated)



The contours of equal dye and suspended solids concentrations plotted on a scaled section of the clarifier graphically illustrate the flow patterns that exist. Principle findings of this phase of the study are summarized below.

Inlet and Outlet Design:

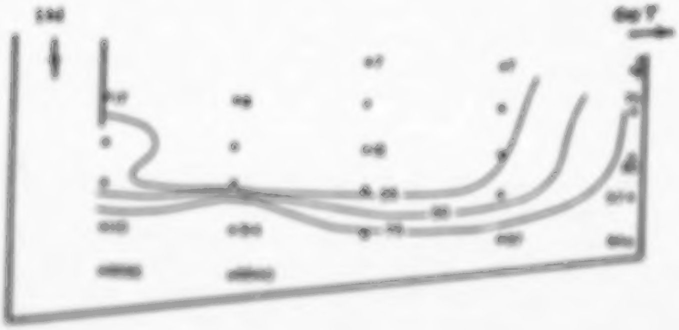
1. For circular clarifiers, the typical solid skirt baffles induce a back eddy region that prevents good vertical distribution of incoming mixed liquor.

2. Effluent weir leveling is critical to good horizontal distribution of flow. Weir level variations of ± 3.0 mm (0.12 in.) can result in flow direction preference of ± 20%.

3. Effluent weir placement at the periphery of circular clarifiers is not optimum. This results from the fact that typical flow patterns are directed radially outward along the bottom of the clarifier above the sludge blanket and then turn strongly upward at the peripheral wall. The peripheral wall where the effluent weir is located is then the location of greatest upward flow and, therefore, the region of highest solids levels. This phenomenon is clearly shown in the radial cross-section of Figure 3.

Figure 3. Distribution of TSS in a Radial Cross-Section of Stamford's Clarifier #1. The Effects of Localized Upflow are Apparent

(Note: Vertical Scale Exaggerated)



Note the high total suspended solids ranging from 78 to 97 mg/L in the immediate vicinity of the effluent weir. It is also obvious that a large center portion of the clarifier surface is not effectively used even though average overflow for the entire surface area is the common design standard for final clarifiers in the U.S.

Effect of Sludge Withdrawal:

For circular clarifiers, settling performance is adversely affected by high rotational speeds of sludge removal mechanisms. In one clarifier, an increase in effluent suspended solids from 25 to 75 mg/L was noted at the effluent weir after passage of the sludge collection mechanism. Excellent correlation was found between these solid waves and sludge collector movement. The solid wave was found to follow the sludge collector by about 90°. At Morganton, North Carolina, reducing the sludge collector speed by 56% reduced total effluent suspended solids from 36.6 to 25.5 mg/L.

Subordinate Flow Influences:

In addition to the above, four additional factors (wind, temperature, seiche, and speed of hydraulic response) were studied. These results are summarized below.

1. Maximum changes in inlet and outlet temperature was 0.06°C and was determined to have no significant influence on clarifier performance.

2. The effect of wind speeds was evaluated for rectangular clarifiers varying between 2-4 meters (7-14 ft) deep by 10-50 meters (33-164 ft) long. The wind speeds necessary to cause a water surface elevation difference of 2 mm (0.08 in.) was 113 km/h (70 mph) for a 10 meter (33 ft) basin, 37 km/h (23 mph) for a 30 meter (98 ft) basin, and 23 km/h (14 mph) for a 50 meter (164 ft) long basin. For typical V-notch weirs, a 3 mm (0.12 in.) water surface elevation difference would cause a 20% difference in flow. The conclusions drawn from this analysis is that wind speed, although not readily observable, can exert a significant influence on clarifier performance by creating a directional imbalance.

3. Surface seiche periods for rectangular clarifiers were calculated to range from 3.2 seconds for a 4 meter (13 ft) deep by 10 meter (33 ft) long clarifier to 25.0 seconds for a 2 meter (7 ft) deep by 50 meter (164 ft) long clarifier. Final clarifiers were noted to have two free surfaces, the visible water surface and the sludge blanket surface. The oscillation period for sludge blankets was judged to be double the time period shown for the visible surface oscillation. Because of the wide variations in tank size and geometry, frequency and amplitude of clarifier seiches, quantitative determination of their overall effect on

clarifier performance could not be made.

4. Hydraulic response speed for clarifiers was found to follow the small amplitude wave speed. For a 3.0 meter (10 ft) deep clarifier, the wave speed was calculated to be 5.5 meters/second (18 fps). A rapid change in influent flow rate will be reflected at the effluent weir in a matter of seconds for most clarifier sizes, thus indicating the importance of reducing influent pumping peaks.

Solids Transport Model:

- By considering horizontal flows in clarifiers to be turbulent, and by using some simplifying assumptions including uniform eddy viscosity and particle settling velocity, a diffusion model was formulated to describe solids transport within the clarifier resulting from amplitude variations in plant flow. The model proposed is shown below.

$$T_R = \frac{1}{T}\sum_{i=1}^{n}\left\{\left(\frac{\tilde{u}}{\bar{v}}\right)^3 \Delta T\right\}_i \qquad (2)$$

where T_R = solids transport relative to that expected at steady flow

T = total period of measured values

$\tilde{u}$ = spatial average instantaneous forward velocity at same cross-section (mid-radius of circular clarifier)

$\bar{v}$ = time average of $\tilde{u}$ values

In one demonstration at Holly Hill, Florida, the installation of new reaction baffles and a 3.8 m^3 (1,000 gal) storage upstream of the final clarifier reduced effluent suspended solids from 20.7 to 15.5 mg/L in one case and from 17.2 to 11.5 mg/L in another. The limited data collected indicated hydraulic turbulence is created rapidly by influent hydraulic peaks, but die-off time is much longer, on the order of basin detention time, thus significantly increasing the time of additional solids transport.

Because of the encouraging initial results, a major follow-on study is planned to further develop and verify the solids transport model. It is anticipated that the results of this investigation of final clarifier performance, along with the follow-on studies, will lead to improved design procedures used by consulting engineers as well as improved standard designs by major equipment suppliers. The demonstrated success in modifying existing clarifiers to improve suspended solids removal during this study at Morganton, North Carolina and Holly Hill, Florida have proven to be very cost effective compared to alternative upgrading methods. The overall study has also indicated the importance of weir leveling, minimizing plant influent pumping peaks, even flow distribution between parallel clarifiers, and maintenance of appropriate rotational speeds of sludge collectors.

Mechanical Components Reliability

Another major effort in EPA's Plant Operation and Design Research Program is development of a new data base on the reliability of major mechanical components. Preliminary results of a study conducted by Southwest Research Institute is summarized below (14).

The primary objective of this project is to quantify the in-service reliability of a broad classification of generic types of critical mechanical components found in conventional secondary wastewater treatment plants. The reliability statistics investigated include mean time between failure (MTBF), mean time to repair (MTTR), and availability as a percent of scheduled operating time. Conventional secondary wastewater treatment processes included in the study are air activated sludge, oxygen activated sludge, trickling filter, and rotating biological contactor treatment systems. The investigative approach taken in this study was to:

1. Select the broad classifications/generic types of critical mechanical components which, upon failure, would have significant adverse impact on effluent quality
2. Identify a representative number of treatment plants within the United States which have good maintenance records for the generic types of equipment of interest
3. Visit selected plants to collect data and statistically analyze and evaluate the results.

The broad classification of equipment selected for this first phase study effort includes:

1. Pumps
2. Power transmissions
3. Motors
4. Compressors
5. Diffusers
6. Valves
7. Controls
8. Pressure vessels
9. Conveyors

The above classification of equipment was based on a criticality analysis of the processes to be included in the study. The analysis included components of the four secondary treatment processes previously listed.

Treatment plants were then contacted to determine the availability of the data included. Nine plants having the records necessary were visited and the available data were collected. The data was then keypunched and the reliability statistics calculated by computer-aided analysis.

Mean time between failure (MTBF) data for power transmissions and motors are presented in Tables 7 through 13. These data were obtained from the computer printout.

Table 7. Mean Time Between Failures (MTBF) by Application

Broad Classification - Power Transmission

Application	(1) No. of Units	(2) No. of Failures	(3) Operation Hrs ($x10^6$)	(4) Overall MTBF ($x10^6$) Hrs	(5) Min. MTBF ($x10^6$) Hrs	(6) Max. MTBF ($x10^6$) Hrs	(7) Overall 90% CL ($x10^6$)
Raw Wastewater Pumping	6	10	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Intermediate Wastewater Pumping	7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Clarifier, Circular Center Drive	65	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Clarifier, Square/Rectangular Center Drive	35	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Return Activated Sludge	27	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Oxygen Generation/Storage	5	0	[illegible]	[illegible]	-	-	[illegible]
Dissolved Air Production	6	2	[illegible]	[illegible]	-	-	[illegible]
Rotating Biological Contactors	56	0	[illegible]	[illegible]	-	-	[illegible]
Mechanical Aerators	82	12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Total	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 8. Mean Time Between Failures (MTBF) by Size Range

Broad Classification - Power Transmission

Size (Horsepower)	(1) No. of Units	(2) No. of Failures	(3) Operating Hrs (x10^6)	(4) Overall MTBF (x10^6) Hrs	(5) Min. MTBF (x10^6) Hrs	(6) Max. MTBF (x10^6 Hrs	(7) Overall 90% CL (x10^6)
0-1	76	243	6.18386	0.02537	0.009929	2.5120	0.0234087
1-5	19	43	0.4806	0.01101	0.009816	0.01785	0.009119
6-25	61	1	2.3100	1.37640	0.63160	1.8670	0.59387
26-100	36	12	0.7426	0.05862	0.03476	0.06091	0.04176
101-500	45	10	0.83474	0.07838	0.007841	0.3764	0.054204
Over 500	12	2	0.55206	0.20645	0.02689	0.1763	0.10373
Total	249	311	11.10386	0.03562	0.01101	1.3764	0.03317

Table 9. Mean Time Between Failures (MTBF) by Generic Group

Broad Classification - Power Transmission

Generic Group	(1) No. of Units	(2) No. of Failures	(3) Operating Hrs. (x10^6)	(4) Overall MTBF (x10^6) Hrs	(5) Min. MTBF (x10^6) Hrs	(6) Max. MTBF (x10^6) Hrs	(7) Overall 90% CL (x10^6)
Concentric Reducer	38	2	[illegible]	[illegible]	[illegible]	0.3764	[illegible]
Parallel Shaft	73	2	[illegible]	[illegible]	[illegible]	1.8670	[illegible]
Right-angle Shaft	51	180	[illegible]	[illegible]	[illegible]	2.5120	[illegible]
Vertical Shaft	53	44	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Variable Speed Drive Hydraulic	13	9	[illegible]	[illegible]	-	-	[illegible]
Variable Speed Drive Others	4	8	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Gear Box	37	65	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Chain Drive	4	5	[illegible]	[illegible]	-	-	[illegible]
Belt Drive	3	5	[illegible]	[illegible]	-	-	[illegible]
Total	249	311	11.10386	0.03562	[illegible]	[illegible]	0.03317

Table 10. Mean Time Between Failures (MTBF) by Application

Broad Classification - Motors
Generic Group - Multiphase

Application	(1) No. of Units	(2) No. of Failures	(3) Operating Hrs. $(\times 10^6)$	(4) Overall MTBF $(\times 10^6)$ Hrs	(5) Min. MTBF $(\times 10^6)$ Hrs	(6) Max. MTBF $(\times 10^6)$ Hrs	(7) Overall 90% CL $(\times 10^6)$
Raw Wastewater Pumping	25	26	0.5997	0.02249	0.00983	0.02795	0.01772
Intermediate Wastewater Pumping	14	2	1.0500	0.3931	0.0422	1.3340	0.1973
Screening/Communition	3	0	1.2160	1.8100	-	-	0.5283
Clarifier, Circular Center Drive	62	20	2.5530	0.1236	-	-	0.09442
Clarifier, Square/Rectangular Peripheral	28	0	3.1224	4.4935	0.5650	2.5120	1.3560
Return Activated Sludge Handling	27	48	1.0227	0.02102	-	-	0.01759
Recirculation Pumping	14	6	0.3541	0.0531	-	-	0.03362
Oxygen Generation/Storage	5	0	0.0564	0.08392	-	-	0.02449
Dissolved Air Production	11	22	0.5255	0.02319	0.01858	0.02282	0.01792
Dissolved Air/O_2 Application	2	2	0.0361	0.01353	0	0	0.006789
Rotating Biological Contactor	56	56	1.2550	0.02215	-	-	0.01876
Mechanical Aerators	24	0	0.4510	0.6711	-	-	0.1959
Secondary Sludge Pumping	14	1	0.2529	0.1513	0.0216	0.3226	0.0650
Total	285	103	12.4948	0.0680	0.02102	4.4935	0.0620

Table 11. Mean Time Between Failures (MTBF) by Size

Broad Classification - Motors
Generic Group - Multiphase

Size (Horsepower)	(1) No. of Units	(2) No. of Failures	(3) Operating Hrs. $(\times 10^6)$	(4) Overall MTBF $(\times 10^6)$ Hrs	(5) Min. MTBF $(\times 10^6)$ Hrs	(6) Max. MTBF $(\times 10^6)$ Hrs	(7) Overall 90% CL $(\times 10^6)$
0-1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1-5	[illegible]	[illegible]	[illegible]	[illegible]	-	-	[illegible]
6-25	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
26-100	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
101-500	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Over 500	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Total	285	103	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 12. Mean Time Between Failures (MTBF) by Size

Broad Classification - Motors
Generic Group - Variable Speed AC

Size (Horsepower)	(1) No. of Units	(2) No. of Failures	(3) Operating Hrs. ($x10^6$)	(4) Overall MTBF ($x10^6$) Hrs	(5) Min. MTBF ($x10^6$) Hrs	(6) Max. MTBF ($x10^6$) Hrs	(7) Overall 90% CL ($x10^6$)
101-500	15	0	0.27100	0.40330	-	-	0.11770
Over 500	2	2	0.03614	0.01351	0.01081	0.01081	0.00682
Total	17	2	0.30714	0.11482	0.01351	0.4033	0.05795

Table 13. Mean Time Between Failures (MTBF) by Application

Broad Classification - Motors
Generic Group - Variable Speed AC

Application	(1) No. of Units	(2) No. of Failures	(3) Operating Hrs. ($x10^6$)	(4) Overall MTBF ($x10^6$)	(5) Min. MTBF ($x10^6$)	(6) Max. MTBF ($x10^6$)	(7) Overall 90% CL ($x10^6$)
Intermediate Wastewater Pumping	1	1	0.01807	0.0181	-	-	[illegible]
Return Activated Sludge Handling	16	1	0.28907	0.1720	0.01081	0.4033	0.07632
Total	17	2	0.30714	0.11482	0.01081	0.1720	[illegible]

Tables 7 through 13 present MTBF values by application, size range, and generic group for the broad classifications of power transmission and motors. The MTBF values are shown in column four, and columns one through three indicate the quantity of data utilized to calculate the MTBF values. Columns five and six show the minimum and maximum MTBF values calculated for the given application, size range, and generic group. These values define the dispersion across the application, size range, or generic group. The last column lists the 90% confidence level MTBF value. There is a 90% chance that the true MTBF value is at least equal to the value shown in the last column. Other statistics have been calculated but are not included in these tables. These include mean time to repair, availability, corrective and preventative maintenance hours/unit/year.

The component MTBF values developed during this study have been compared with other reliability data sources. They appear to be in the same order of magnitude. This would indicate that they are reasonable estimates of the true MTBF.

These data will provide valuable information to design engineers and plant operations personnel. Of greater significance, however, is that the performance of generic equipment now can be predicted for a given application, size range, and unit operation. Proper equipment selection for a given application can also be enhanced. Operations personnel can benefit in that preventative maintenance efforts can be more efficiently managed.

Systems Reliability Analysis

The four year goal of USEPA reliability research program is to integrate the findings of the previously described research efforts in biological process stability/reliability and physical process and mechanical component reliability into a comprehensive reliability design methodology. The design methodology will permit trade-off analysis of total treatment works costs vs. various levels of effluent or unit process reliability. Results will be used to establish new reliability guidelines to indicate desired equipment redundancy and to identify critical spare part inventories for treatment plants. Development of the reliability model will begin in 1981, and is scheduled for completion in late 1982.

It is expected that use of the systems approach to reliability analysis will reduce conservative design features of certain municipal treatment plant components that are not critical in consistently meeting effluent discharge standards.

Plant Operation and Management

Both local governments who bear the cost of municipal treatment plant operations in the U.S. and state and USEPA who share the responsibility for enforcement of effluent standards are concerned with the rising labor and energy costs of municipal plant operation and management.

USEPA programs, in the past, have addressed the cost of operation as a part of the cost effectiveness analysis required in the Construction Grants Program, but have not aggressively dealt with improving plant operation and management until passage of the Clean Water Act of 1977. Operations and management concerns have become more acute recently because of the increasing labor and energy costs, increased federal enforcement activities, and the trends toward decentralized treatment by an increasing number of smaller facilities. The Office of Research and Development has undertaken a number of studies designed to improve the management and lower the cost of operation at municipal treatment facilities. These efforts include: (1) development of a national computerized operation, maintenance, and management data base to collect, store, and analyze O&M costs of municipal treatment facilities throughout the U.S.; (2) development of a circuit rider operation and maintenance cost model (CROM); and (3) the increased use of instrumentation and automation to optimize plant operation and control. Each of these recent activities is briefly described below.

Computerized Operation, Maintenance, and Management Data Base

The computerized data base was established to provide a centralized source of biological wastewater treatment data related to design, cost, operation, and performance of municipal treatment facilities and to allow the extraction of data in an efficient reporting format which permits data manipulation without major programming involvement. When fully functional, the system will represent one of the most detailed and comprehensive sources of engineering and performance data that will be capable of supplying the information necessary for analyses and decision making and will facilitiate the capability of the research program to respond to federal, state, and local agency requests for information and assistance.

In order to optimize computer data storage utilization and the process of information extraction, the Wastewater Treatment Processes Information System, as the computerized data base is formally termed, was designed and developed in four subsystems.

1. Data Management Subsystem
   All data are entered through the Data Management Subsystem. It permits data to be added to and changed within the data Master File and to be checked for proper value limits and coding. Several functions are included to optimize use of storage space and to allow maximum efficiency in data retrieval.

2. Data Extract Subsystem
   The Data Extract Subsystem consists of COBOL programs to extract the desired data from the master file. The individual data elements extracted are stored in an Extract File. The outputs from this subsystem are used by the report generation and statistical analysis subsystems.

3. General Reporting Subsystem
   An easily programmable Mark IV report generator comprises this subsystem. Through Mark IV, a programming language that requires a low level of programming effort, a variety of reports can be generated using the data elements in the Extract File. Data presented through the subsystem may be organized in detailed or summarized format using some calculations.

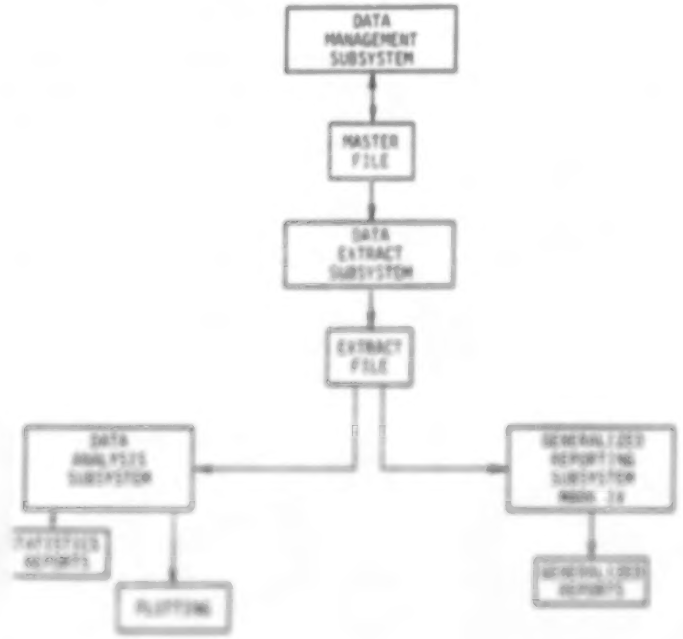
4. Statistical Analysis Subsystem
   This subsystem performs statistical analyses not capable on Mark IV and includes plotting capabilities on a drum plotter.

The relationships of these subsystems are shown on the system flow chart in Figure 4.

Inputs to Computerized Data Base

Data inputs to the computerized data base originate in various forms from rough operating logs to centralized computerized data systems. The data are of varying levels of quality ranging from precise research data to self monitoring data generated by municipalities. To identify the quality level, one of three quality factor notations is assigned to each data element when it is entered into the Master File.

Figure 4. System Flow Chart for "Wastewater Treatment Processes Information System"


DATA MANAGEMENT SUBSYSTEM
MASTER FILE
DATA EXTRACT SUBSYSTEM
EXTRACT FILE
DATA ANALYSIS SUBSYSTEM
GENERALIZED REPORTING SUBSYSTEM MARK IV
STATISTICS REPORTS
PLOTTING
GENERALIZED REPORTS

The data include design parameters, state/county and permit information, operation costs, current physical conditions, physical plant details, cause and effect and operational factors data, sludge handling and disposal data, and relevant performance information.

The following general data sources have been identified and input to the system during the first year of operation:

1. Summary research reports containing more than 120,000 data elements summarized from other reports, plant operation and performance log sheets, unpublished research data, etc. The reports include performance, monitoring data collected at various time intervals such as hourly, daily, monthly, and/or annually; operational data, O&M cost data, poor plant performance cause and effect data, etc.

2. Research reports on individual plants containing 15,000 general and parameter data elements with a major emphasis on overall plant performance. Operation and performance of individual unit processes may be addressed but usually not in depth.

3. Research reports on 75 facilities with comprehensive data on individual unit processes. These may include a complete evaluation of only one unit such as an aerator, clarifier, etc., or they may include several units. These data elements number about 25,000.

4. Research reports on single parameters such as phosphorus where data is collected on several unit processes both before and after using various biological and chemical means of removing phosphorus. This data will normally be collected for several monitoring points at various time intervals.

5. EPA regional permit and compliance reports on individual plant performance and regional surveillance and analysis reports.

6. EPA Needs Survey which represents 130,000 general and parameter data elements from 10,000 municipal facilities. These data are currently being entered into the data base.

7. EPA Form 7500-5 which includes a range of general and parameter data of low detail on the operation and maintenance of individual wastewater treatment plants.

8. National Survey results for all site visits and preliminary evaluations containing general and parameter data in great detail.

Output Reports

From the data in computerized storage, a large variety of information can be extracted, coordinated, and analyzed in a wide variety of calculations, plots, and reports. Typical programs that have been developed are summarized below.

1. Search the entered data bases and present summary information and statistical calculations of summary information for the various biological treatment plants or a unit process modification as a function of detailed design, operational, loading or performance factors, size, age, location, or performance.

2. Provide a complete breakdown of all influent and effluent constituents and their concentrations for all plants or particular types of plants.

3. Identify all monitoring points and the concentration of each parameter for these points at various time intervals (if given any one time value, the computer shall be required to calculate other time values).

4. Calculate percent removals for all plants or particular types of plants at various flow ranges, temperatures, time of the year, or influent concentrations.

5. Present unit O&M costs as a function of plant size, location, staffing, temperature, etc., for all biological plants and/or modifications thereof.

6. Present overall plant and unit performance as a function of plant size, location, hydraulic and organic loading, staffing, plant operation, temperature, or design factors.

7. Present cause and effect of poor plant performance as a function of plant size, location, hydraulic and organic loading, staffing, plant operation, temperature, design factors, or available funds.

This data base contains detailed O&M cost and performance information from over 300 facilities and less detailed information on 10,000 facilities is now being entered into the program. The data base is expected to grow by inclusion of detailed cost and performance information for 300-500 new facilities per year. The program is being used by both research scientists within EPA's research organization and by state and federal operating program personnel.

Centralized Management of Small Satellite Facilities

In further efforts to study methods to improve overall performance at current operating cost, or to lower overall costs while maintaining acceptable effluent quality the concept of centralized management of satellite treatment facilities is being investigated. On-going studies have included examination of several modes of operation including independent operation of each plant, circuit rider operation, telemetry operation, plant consolidation, as well as various mixes of the above operational modes.

A Circuit Rider Operation and Maintenance Model (CROM) has been developed by EPA researchers based on the detailed O&M data from 14 small treatment facilities in Houston, Texas with flows ranging from 6 to 3,800 m^3/d (0.0016 to 1.0 mgd). Mathematical cost functions were developed as a function of actual plant flow. Separate costs for materials and labor are calculated for operation, maintenance, administration, and circuit riding. The cost for operation includes plant operation, sludge removal, and laboratory recordkeeping. Maintenance costs include maintenance and repair. Administration costs include supervision and management fees. Circuit rider costs include labor and vehicle costs associated with transportation.

Use of the model includes varying any of the above input cost functions to determine the effect on overall system costs. Future development and expansion of the model will permit optimization of the various management modes previously mentioned. Tables 14 and 15 list the input variables and output parameters of the program. In a comparison of the estimated cost of operation and management for 14 plants in Houston, the CROM predicted a circuit rider management mode cost of $234,641 per year compared to an actual cost of $243,737 per year. Refinements to the cost curves used in the model are expected to improve the accuracy substantially.

Future simulation studies are planned in conjunction with management modes that include use of remote sensing and telemetry from satellite facilities to central control stations.

Another EPA sponsored study is currently underway in Cuyahoga County, Ohio, to provide data on the cost and effectiveness of centralized management of a system of small (package) treatment plants using low cost instrumentation, electronics, and telecommunications equipment. The objectives are to evaluate O&M management alternatives and to evaluate the application of micro-electronic technology in reducing costs and optimizing the operation and maintenance of package plants. The study is providing considerable data on the costs and benefits of purchasing, installing, operating, and maintaining monitors on wastewater treatment systems equipment.

The demonstration site is a 760 m^3/d (0.2 mgd) activated sludge package plant operated by the Cuyahoga County Sanitary Engineering Department. The monitoring equipment, including both field proven sensors and safety-security sensors, is oriented toward a basic low-cost monitoring concept capable of providing an alarm at a central site in order to detect impending plant upset or mechanical breakdown. The system is to be optimized for centralized management of small satellite plants that use circuit rider operators. The monitoring system installed is shown in Table 16. It includes sensors and alarms for monitoring equipment status, process operation, and security and safety.

The heart of the monitoring system is the Programmable Logic Controller (PLC) that serves as the interface between the sensors and the telephone alarm transmitting system. The PLC scans through the 23 equipment, process, and security sensors and responds to an alarm status according to a pre-set program of relay-type logic. In alarm status, a telephone call is initiated

Table 14. Input Variables to CROM

NCASE	number of data cases to be executed by the program
NP	number of plants in the circuit rider system
NDATA	number of input data sets to be run for the circuit rider system
ID	alpha-numeric identification for the circuit rider system
NAME	plant name
Q	actual plant flow, mgd
ODHR	hourly labor rate for operations labor, $/hour
MDHR	hourly labor rate for maintenance labor, $/hour
ADHR	hourly labor rate for supervisory labor, $/hour
CRDHR	hourly labor rate for circuit riding labor, $/hour
PCT	indirect labor costs, percentage
WPI	current wholesale price index for industrial commodities (U.S. Deparment of Commerce)
MPH	average travel speed of circuit rider vehicles, miles/hour
CMILE	cost per mile of circuit rider operation, $/mile
CIR	number of circuits or sub-circuits in the system
NSTOP	number of stops on a circuit
MILES	total length of a circuit, miles
DAYS	frequency of a circuit, days/week
WEEKS	duration of a circuit, weeks/year
NS(J)	identification numbers of plants on a circuit

Table 15. Output Parameters from CROM

N	plant identification number
PLANT	plant name
Q	actual plant flow, mgd
MYR	circuit rider miles of travel charged to a plant, miles/year
OMAT	annual cost of operations material, $/year
OLAB	annual labor cost of operations, $/year
MMAT	annual cost of maintenance materials, $/year
MLAB	annual labor cost of maintenance, $/year
AMAT	annual cost of administration material, $/year
ALAB	annual cost of supervision labor, $/year
CRMAT	annual cost of circuit rider material, $/year
CRLAB	annual labor cost of circuit rider labor, $/year
TCOST	annual total cost of plant, $/year

Table 16. Small Plant Monitoring System

FUNCTION	SENSOR	LOCATION	ALARM	PROTECTION AGAINST
ES	Liquid Level	Lift Station Wet Well	Low Level High Level	Loss of Pump Prime Flooding
ES	Power Demand	Lift Station Pumps	On-Off	Power Outage
ES	Thermocouple	Lift Station Pumps	High Temp.	Loss of Lubrication
ES	Air Pressure	Blowers	Low Pressure	Leak in Air Headers
ES	Thermocouple	Blower Bearings	High Temp.	Loss of Lubrication
ES	Power Demand	Blowers	On-Off	Power Outage
ES	Level Detectors	Clarifiers	High Sludge Level	Washout of Sludge
ES	Thermocouple	Return Sludge Pumps	High Bearing Temperature	Loss of Lubrication
ES	Power Demand	Return Sludge Pumps	On-Off	Power Outage
ES	Thermocouple	Sludge Scraper Drive	High Torque	Motor Overload
ES	Power Demand	Sludge Scraper Drive	On-Off	Power Outage
PO	Dissolved Oxygen	Aeration Tanks	Low DO High DO	Shock Load Demand Excessive Aeration
PO	Residual Chlorine	Chlorine Contact Tank	High Level	Excessive Usage
ES,S/S	Chlorine Gas	Equipment Building	High Level	Gas Cylinder Leak
S/S	Smoke/Fire	Equipment Building	On-Off	Fire
PO	Turbidimeter	Effluent	High Level	Process Upset

ES - Equipment Status
PO - Process Operation
S/S - Safety/Security

to reach the circuit rider operator and a pilot light is set for the alarm and recorded on the PLC readout unit. The telephone calling electronics includes a pre-established hierarchy of telephone numbers that repeats in sequence until an electronic acknowledgement signal, actuated by the call receiver, is received by the unit.

The equipment is in place and, although rigorous data generation has not yet begun, some interesting observations have been noted.

1. When chlorine cylinders are changed, an alarm and/or warning condition exists in the cylinder storage area.

2. The chlorine feed unit malfunctioned, requiring a service call by the manufacturer.

3. The bearing temperatures on the blowers ranged from 43°C (110°F) to 49°C (120°F), well within the manufacturer's suggested maximum operating temperature for the lubricant of 71°C (160°F).

4. One of the three final clarifiers is most heavily loaded, consistently showing the highest sludge blanket level and sludge particle washout, particularly during morning peak flows.

5. The plant has only limited flexibility for blower operation in that at least two of the four blowers are required to run the air lift sludge pumps; and in winter, three blowers must often be run to prevent icing in the aerators.

6. A snow plow was stolen from the grounds.

REMAINING PROBLEMS AND FUTURE O&M INITIATIVES

From a national perspective, the record of compliance for municipal treatment plants in the U.S. must be improved to meet the goals of the Clean Water Act of 1977. EPA's Plant Operation and Design research program, as well as research conducted elsewhere, has provided a solid technological basis for improved municipal compliance. Inherent problems in achieving significant plant performance improvements on a nationwide basis continue to be more institutional and attitudinal than technological. The split in responsibility between the designer and operator, together with the lack of professional concern on the part of the designer for successful plant operation over the life of the facility, is the fundamental reason for poor plant performance.

USEPA's efforts, which have been construction oriented, have lacked a directed and effective O&M objective during the Construction Grants Step I, II, and III processes. The scope and emphasis of a new strategy must include the encouragement and support of the development and implementation of rational design standards for conventional and alternative technologies; the development of public and private expertise to plan, design, and review O&M requirements for plant performance optimization; and the development of programs to assure optimum plant operation followed by decisive actions, including enforcement, to assure implementation of the programs.

State and local agencies have not recognized nor accepted their responsibilities for the correction of operational problems for assuring continuing efficient plant operations. These agencies have not sought the assistance of private sector expertise capable of providing operations technical assistance nor have these agencies trained their own personnel in that area. Enforcement activities have not provided the incentives to encourage the development of private sector expertise in operational technical assistance.

The failure of both the public and private sectors of the pollution control community to recognize their responsibilities and to be held accountable for their decisions and actions significantly contribute to the continuing problem.

The engineering design community and equipment manufacturers and suppliers play key roles in the pollution control effort. Yet in providing their services they are not held responsible for the decisions they make in the design of facilities, for the reliability and operability of the facilities and equipment they furnish, nor for the information they disseminate to plant administrative and operations personnel. As a result, the local community is put in the untenable position of spending a lot of money for a treatment plant that may be poorly designed or is incapable of being operated properly. There will be no improvement in the situation until consultants and equipment suppliers are held responsible for their products, the decision they make, and the actions they take.

The designer's responsibility must be extended to include a guarantee of plant performance. The concept that successful plant operation is a professional activity and responsibility must be encouraged.

Improved system design of conventional technology and cost effective design of innovative and alternative processes must be encouraged. Deficiencies in treatment plant design have been identified as one of the primary causes of poor plant performance. Deficiencies are noted not only in the design of individual unit processes but also on the relationship of units within a system that hinders the reliability and stability of the system and prevents the system from performing optimally. The design community has relied upon existing and often irrational design criteria, ignoring comprehensive analysis and performance and operational experience of existing municipal treatment plants.

Existing programs to train operators and provide them the required information to properly monitor, control, and operate their plants are ineffective. Operators have not been able to transfer their classroom instruction to practical on-site problem solving; they do not use much of the guidance and instructional materials provided by federal programs because they do not understand them, and they are provided with often-conflicting information from a variety of sources that are not held accountable for the instruction provided. There is a lack of programs that can improve plant operator on-the-job effectiveness including on-site training and site specific instructional material for process operation and control.

Evaluations of facility plans and design do not adequately consider the instrumentation and automation alternatives to process monitoring and control nor the cost and performance advantages of centralized management of POTWs where feasible as an alternative to construction of regional facilities. There are no requirements for planners and engineers to evaluate alternative strategies to achieve improvements in reliability, energy conservation, operating cost, and efficiency for individual plants or groups of plants.

There is a need for a two-prong approach to insure optimum plant performance through improved operation. First, existing plants being upgraded must undergo a comprehensive O&M review through an improved construction grant review process. Comprehensive plant evaluations should be required for all plants requesting federal grant funding. For plants not needing construction but not in compliance, mandatory O&M reviews should be required as the first step in enforcement actions. Despite existing measures to assist communities to plan for and implement sound O&M programs, there is widespread failure on the part of the community to fulfill its responsibility. The USEPA must provide, through rigorous enforcement actions, the incentive for communities to bear the prime responsibility for fulfilling their obligations.

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STATUS OF DEEP SHAFT WASTEWATER TREATMENT TECHNOLOGY IN NORTH AMERICA

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INTRODUCTION

Municipal pollution control technology is entering a period of rapid change as a consequence of the Clean Water Act Amendments of 1977 (Public Law 95-217) (1) related to innovative and alternative technology. For qualifying technology, these Amendments provide an increased level of Federal funding of up to 85 percent of the capital costs and, if necessary, 100 percent of the replacement costs as a warranty provision. These economic and risk assumption incentives by the Federal government are intended to encourage increased consideration of lower cost and more energy efficient municipal wastewater treatment facilities and accelerate the rate at which such technologies are developed and put into practice. A technology can qualify as innovative if it offers, compared to the most cost effective conventional technology, the potential to reduce life cycle costs by 15 percent or reduce net primary energy requirements by 20 percent. These cost and energy effectiveness comparisons must be made on a site specific basis. Therefore, no technology, per se, can be deemed innovative. Specific technologies have been deemed alternative. A complete description of the criteria and methodology of establishing whether a technology is innovative or alternative in a specific application is included in the Innovative and Alternative Technology Assessment Manual (2).

One of the emerging technologies that will be evaluated against innovative criteria in the next few years is the Deep Shaft Process developed by Imperial Chemical Industries Ltd. (ICI) in the United Kingdom and Europe and refined and marketed in North America by its Canadian affiliate C-I-L Inc. The process is marketed in Japan by ICI Japan.

Based on published information, this technology provides several potential process improvements, compared to conventional activated sludge technology, which may result in savings in treatment costs, energy requirements, and space requirements. These potential economies are attributed to substantially higher organic loading capacity, reduced detention time, more efficient oxygen and power utilization through improved contacting, and reduced sludge production and processing requirements.

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The purpose of this paper is to briefly review the development of Deep Shaft technology, report on the initial results of a U.S. Environmental Protection Agency (EPA) demonstration evaluation project at Ithaca, New York, and report on the status of the technology in North America.

HISTORICAL DEVELOPMENT

BACKGROUND

The ICI Deep Shaft Process originated from basic aerobic fermentation technology developed by the Agricultural Division of ICI for the production of single cell protein from methanol (2,3). This fermentation technology utilizes a pressure cycle contactor in which air for biological oxidation also provides the driving force for liquid circulation. Reportedly (4), the fermenter is capable of achieving an oxygen transfer intensity of 10 kg O_2/m^3/hr (0.62 lb/ft^3/hr) at a power economy of 1.5 kg O_2/kWh (2.5 lb/hp-hr). Oxygen transfer efficiency at this intensity is over 50 percent. A schematic of the ICI single cell protein fermenter and a description of British and Japanese developmental efforts can be found in Reference 5.

Municipal wastewaters and even high strength industrial wastewaters do not require such a high oxygen transfer intensity as that required for single cell protein production because of lower concentrations of microorganisms and substrate, less biodegradable substrate, and slower microorganism growth rates in waste treatment applications. For these reasons, the fermenter configuration was changed for waste treatment applications to provide much longer bubble contact time by increasing the length of the reactor in the form of a deep shaft to achieve higher oxygen transfer efficiency and power economy. Another significant difference between wastewater treatment applications and fermenter applications is the cell mass management objective to one of minimization rather than maximization.

PROCESS DESCRIPTION

A simplified schematic of the basic Deep Shaft reactor is shown in Figure 1. Raw wastewater, after normal screening and degritting, is continuously discharged into the downcomer together with the return sludge. Compressed air is piped into both the downcomer and the riser to provide the oxygen required for treatment as well as the driving mechanism to maintain circulation velocities of 1-1.5 m/sec (3-5 ft/sec). The depth of the shaft is set so almost all of the bubbles are dissolved at the bottom.

From the downcomer, aerated liquid passes into the riser section and flows upward. As the hydrostatic pressure decreases, bubbles form removing carbon dioxide, nitrogen, and unused oxygen from the liquid. A slightly expanded section at the top of the shaft (head tank) serves as a gas disengagement zone. Some of the mixed liquor overflows the head tank to a solids separation process and the remainder of the mixed liquor is recirculated to the downcomer. The details of the reactor design and mode of operation vary depending on the choice of sedimentation or flotation for the final solids separation function. These details are provided in subsequent portions of the paper.

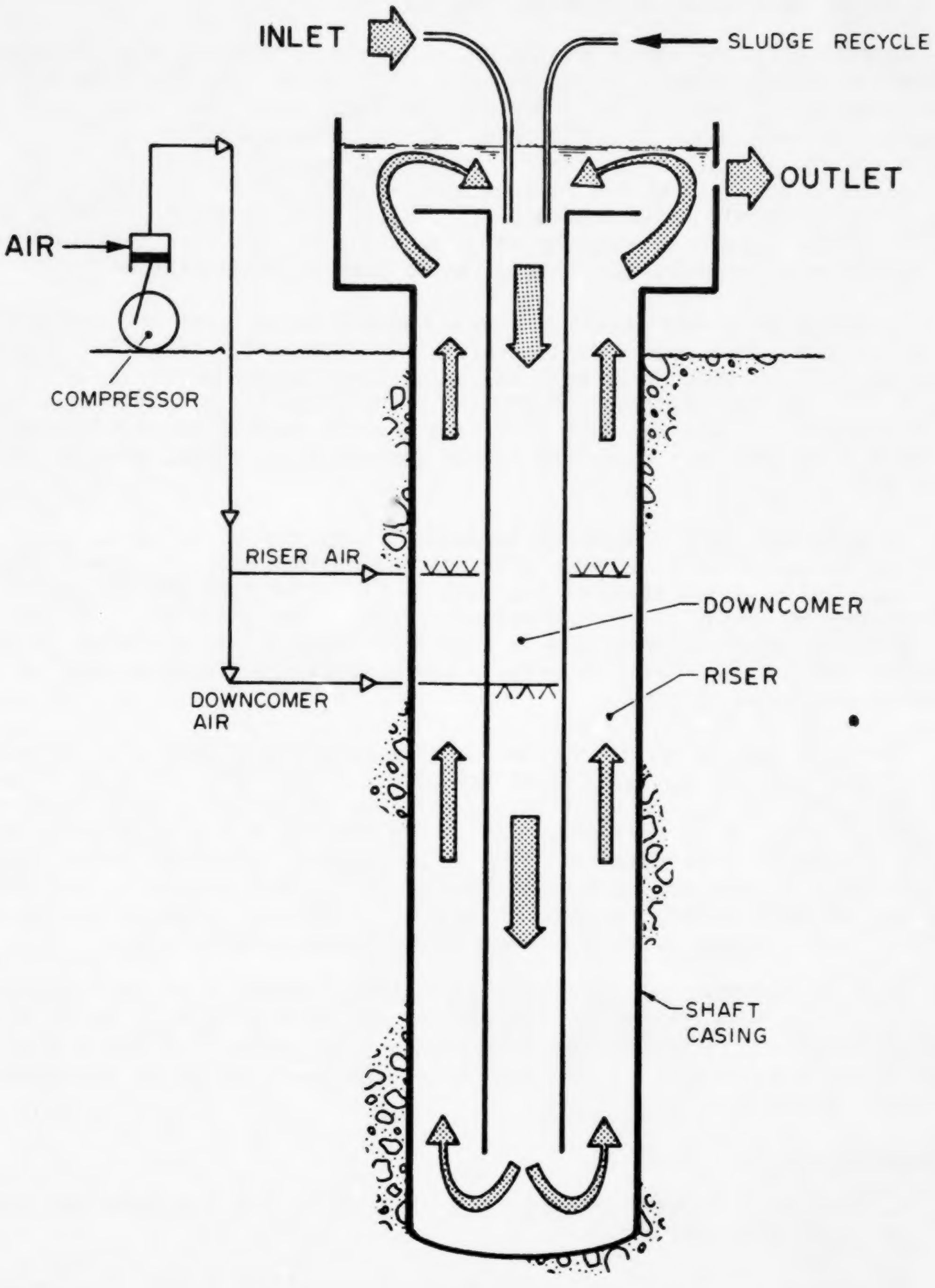
INLET
SLUDGE RECYCLE
OUTLET
AIR
COMPRESSOR
RISER AIR
DOWNCOMER
RISER
DOWNCOMER AIR
SHAFT CASING

FIGURE 1. ICI SHAFT CONFIGURATION AND HYDRAULICS

FIRST PILOT FACILITY AT BILLINGHAM, ENGLAND

In mid-1974, evaluation of the ICI pilot facility was started at the Billingham sewage works in Great Britain. A total of 363 m^3/day (96,000 gpd) of wastewater was treated in a Deep Shaft reactor 39 cm (15.25 in.) in inside diameter (ID) and 130 m (426 ft) deep. The solids separation sequence included natural flotation, mechanical degassing, and sedimentation. Flotation was chosen to make use of the fine bubbles already present in the rising liquid and to permit rapid return of sludge. Flotation also provides a thickened float for subsequent dewatering. Degassing was provided to eliminate the interference residual air bubbles would have on sedimantation.

The pilot unit, when evaluated on a domestic waste with influent BOD_5 and suspended solids concentrations of 195 and 210 mg/l, respectively, produced an effluent BOD_5 of 28 mg/l and an effluent suspended solids of 36 mg/l at a 1.3-hr nominal shaft detention time. The mixed liquor volatile suspended solids (MLVSS) content was 3100 mg/l, and the F/M loading was 1.1 kg BOD_5/day/kg MLVSS. The effluent suspended solids concentration ranged from 18 to 55 mg/l (6).

In September 1975, the solids separation sequence of the pilot plant facility was modified to improve effluent quality and simplify operations. This amounted to elimination of the natural flotation tank and the use of a vacuum degasser prior to the sedimentation tank. The pilot plant at Billingham is approximately represented in Figure 2. Figure 2 is actually a schematic of the ICI Deep Shaft Process in the gravity clarification mode as it is being evaluated at Ithaca.

The pilot unit at Billingham was evaluated in the gravity clarification mode at constant and variable flows for almost a year (7). Constant flow results showed average effluent values for BOD_5 of 15 mg/l (range, 6-35) and suspended solids of 18 mg/l (7-39) at an F/M loading of 0.8 kg BOD_5/day/kg MLVSS, a nominal shaft detention time of 70 minutes, and mixed liquor suspended solids (MLSS) of 4400 mg/l (2000-6000). Variable flow results indicated average effluent values for BOD_5 of 22 mg/l (9-50) and suspended solids of 26 mg/l (5-62) at approximately comparable operating conditions.

Sludge production was carefully monitored at Billingham under constant flow conditions and with an average F/M loading of 1.26 kg BOD_5/day/kg MLVSS. A yield of 0.5 kg sludge/kg BOD_5 converted was suggested as a reasonable level to anticipate with typical raw wastewater characteristics and under diurnal flow operation (7).

DEVELOPMENT IN NORTH AMERICA

In 1975, ICI extended process licenses to C-I-L Inc. in North America and ICI Japan in Japan.

Eco-Research Ltd., a wholly-owned subsidiary of C-I-L Inc., is responsible for Deep Shaft development in North America. Eco established a pilot plant in Paris, Ontario, Canada, to optimize the ICI Deep Shaft flowsheet incorporating flotation rather than sedimentation as the means of suspended solids separation.

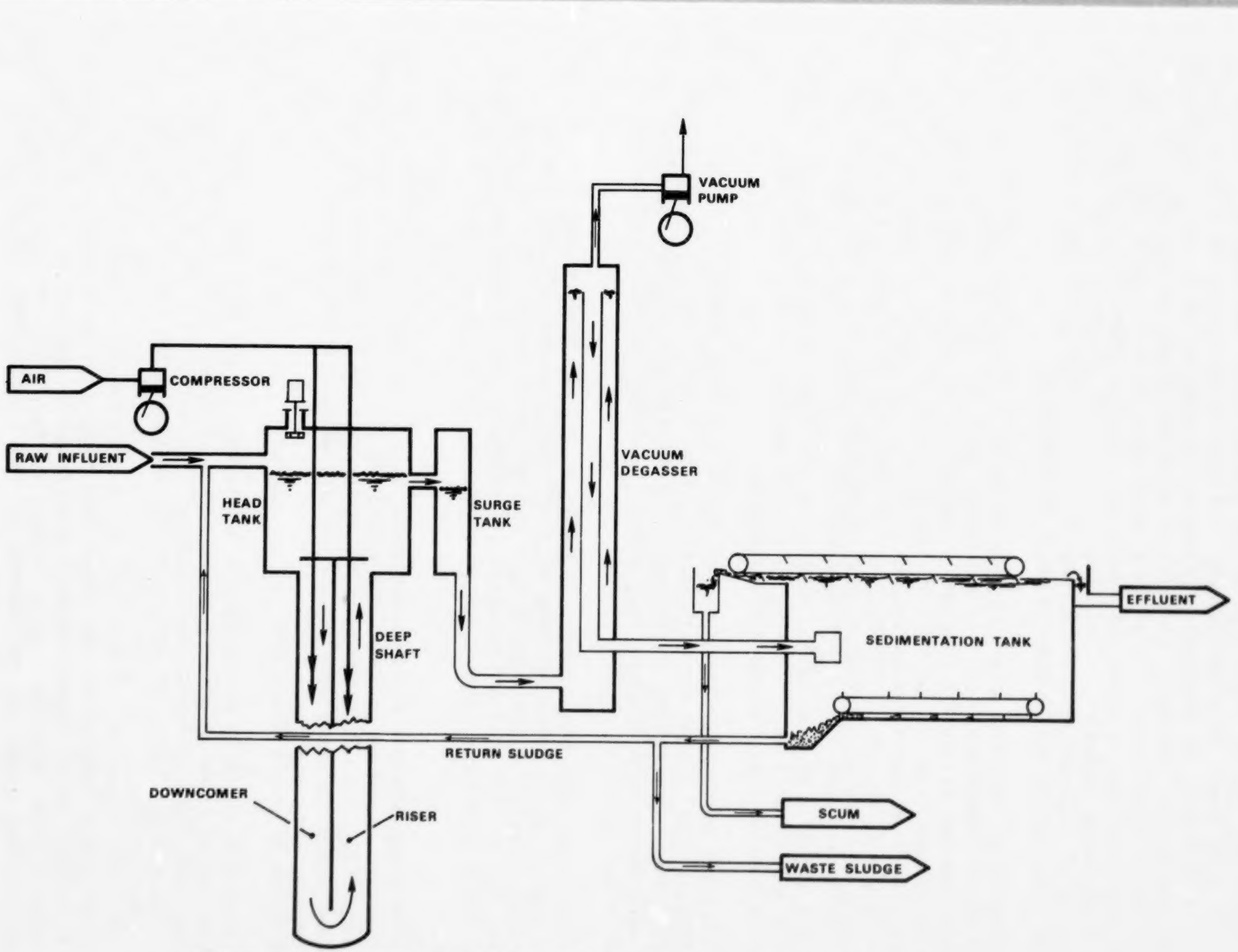
VACUUM PUMP
AIR
COMPRESSOR
RAW INFLUENT
HEAD TANK
SURGE TANK
VACUUM DEGASSER
DEEP SHAFT
SEDIMENTATION TANK
EFFLUENT
RETURN SLUDGE
DOWNCOMER
RISER
SCUM
WASTE SLUDGE

FIGURE 2. SCHEMATIC OF DEEP SHAFT SYSTEM USING GRAVITY CLARIFICATION

Deep Shaft with gravity clarification was thought to be uneconomical in North America. Deep Shaft aeration followed by flotation clarification offered the potential to reduce the bioreactor volume and to make multiple use of compressed air for biological treatment, circulation of mixed liquor through the shaft, and flotation separation. Flotation allowed the bioreactor to be operated at MLVSS concentrations in excess of 10,000 mg/l without impacting suspended solids separation using flotation. Flotation clarification yields a solids float concentration of 5-10 percent, which reduces return sludge volume and eliminates the need for additional thickening prior to dewatering.

Results of the pilot plant studies at Paris on 454 m^3/day (120,000 gpd) of a combined domestic-industrial wastewater were reported by Sandford (7). Influent average values for BOD_5 and suspended solids were 141 and 217 mg/l (75-726), respectively. Ten mg/l of liquid ferric chloride is added to the raw wastewater at Paris for phosphorus removal. The shaft at Paris has an ID of 39 cm (15.25 in.) and a depth of, 155 m (508 ft), providing a nominal detention time of 55 minutes. With an average MLSS of 6000 mg/l (3000-16,000), the F/M loading averaged 1.0 kg BOD_5/day/kg MLVSS (0.4-2.0). The flotation area provided was 13.9 m^2 (150 ft^2). Because of the high surfactant load of 30 mg/l average and 150 mg/l peak from the industrial wastes, 10 mg/l of polymer was required to stabilize flotation clarification.

Effluent quality for the period of March-June 1977 averaged 24 mg/l (7-96) for BOD_5, 15 mg/l (1-56) for soluble BOD_5, and 29 mg/l (10-121) for suspended solids. Weekend operating results with a lower (30 percent COD) industrial load were better with effluent BOD_5 values of 13 mg/l (5-33) and suspended solids values of 18 mg/l (10-32).

Short-term operating results (1 month) at a flow of 681 m^3/day (180,000 gpd) and a nominal shaft detention time of 35 minutes produced effluent quality values of 23 mg/l for BOD_5 and 28 mg/l for suspended solids. For a 2-week period, the textile mill and associated surfactant contribution was eliminated. Under these conditions, stable operation was achieved with a polymer dose of 3 mg/l.

Operating problems at Paris during June 1977 included power failures (8 hr) and blockage in the surge tank feed system (16 hr). Development efforts through the late 1970's were directed at eliminating the need for chemical flotation aids and improving operational stability of flotation clarification.

IMPORTATION TO THE UNITED STATES

From 1975 to 1978, EPA scientists were aware of the Deep Shaft pilot studies taking place at Billingham and Paris. Numerous meetings and consultations took place with representatives of ICI, C-I-L Inc., and Eco-Research Ltd. The primary topics of these meetings concerned the optimum time, place, and mechanisms for defining the potential applicability of Deep Shaft technology to United States (U.S.) municipal wastewater treatment practices. It was mutually agreed that the process would have restricted market potential in the States without an EPA sponsored evaluation project. Because of the desire of the several firms to protect their collective and respective patent positions, it was further agreed that no research and development (R&D) project using public funds should be initiated in the U.S. until such time as the technology was developed to the point where it could justifiably be demonstrated at a meaningful engineering scale.

After preliminary discussions with several municipalities, the City of Ithaca, New York, was selected as an acceptable site for conducting a comprehensive evaluation. Effective September 21, 1978, an EPA demonstration grant (No. S806801) was awarded to Ithaca to construct and operate a 757-m^3/day (0.2-mgd) Deep Shaft pilot demonstration plant on the grounds of its existing wastewater treatment plant. The grant project is discussed in detail in a subsequent section of this report.

RECENT TECHNOLOGICAL DEVELOPMENTS

BACKGROUND

Eco-Research Ltd. has conducted a continuing program of Deep Shaft research and development since its parent firm, C-I-L Inc., acquired the North American license to the technology in 1975. As indicated in the previous section, Eco's early efforts were devoted to evaluating the feasibility of mating a Deep Shaft bioreactor with flotation clarification for liquid/solids separation. The resulting flowsheet is referred to as Eco I.

Eco I utilizes the basic shaft configuration developed by ICI. In this configuration, influent wastewater and return sludge are added to the head tank surmounted on the shaft. Mixed liquor is drawn off from the side of the head tank for subsequent clarification.

Once the feasibility of employing flotation for independent mainstream clarification became evident, Eco turned its attention to modifying the design and operation of the shaft to enhance its biological profile and improve its compatibility with the revised mode of solids separation. Examination of Figure 1 reveals that the upper section of the shaft downcomer is not aerated, except for residual dissolved oxygen (DO) carrying over from the riser. The combined mixture of influent wastewater and return sludge introduced at the top of the downcomer in the ICI and Eco I configurations does not contact virgin DO until it reaches the downcomer air injection point at approximately the 40 percent depth level of the shaft. Consequently, as much as 15-20 percent of the total shaft circulation route may be anoxic, depending on waste strength. It is apparent that the upper section of the downcomer leg is not the most effective section of the shaft for satisfying the high initial oxygen demand of the substrate.

A feature of the ICI and Eco I shaft operating modes that is counterproductive to efficient flotation is the withdrawal of mixed liquor at the surface. As mixed liquor travels up the riser leg, it degases. By the time it reaches the head tank, much of the carbon dioxide, nitrogen, and unused oxygen, which were dissolved in the lower section of the shaft, has been stripped from solution. The dissolved gas driving force for inducing flotation clarification of mixed liquor solids is thereby considerably reduced.

SHAFT MODIFICATIONS - ECO II

The above described characteristics of ICI and Eco I shaft geometry and operation led to the development of a revised shaft configuration (Eco II) with influent injection and mixed liquor takeoff both accomplished at depth. As illustrated in Figure 3, Eco II utilizes a multi-channel design concept featuring one primary downcomer (D_1), one primary riser (R_1), one secondary downcomer (D_2), and three secondary risers (one designated R_2 and two designated D_2R_1). The secondary downcomer and secondary risers are restricted to the upper 35-50 percent of the shaft. The primary downcomer and primary riser run the full depth of the shaft. In the lower section of the shaft, the primary riser occupies the full annulus between the primary downcomer and the outer casing. In the upper section in the area of the secondary downcomer and secondary risers, the primary riser consists of two channels occupying only a portion of the annulus.

Influent wastewater and return sludge are introduced to the shaft through the D_2 secondary downcomer. The D_2 downcomer is turned up at its lowest point, forcing the influent-return sludge mixture back up the two short D_2R_1 secondary risers and on into the two upper R_1 primary riser channels. The injection level where the D_2R_1 sections discharge into the upper primary riser channels is normally selected to fall within the range of one-third to one-half the shaft depth.

Compressed air is injected into the upturned D_2R_1 risers near the bottom. Influent wastewater and microorganisms are thereby contacted with makeup air at a point of high oxygen solubility. The two upper channels of the primary riser now become the zones of initial high oxygen uptake, replacing the less efficient upper downcomer section which serves this role in the ICI and Eco I shaft designs.

Air injection into the D_2R_1 secondary risers serves two purposes. First, it aspirates the influent wastewater-return sludge mixture into the shaft at depth using the air-lift principle. Second, it provides the motive force (along with primary downcomer air) for circulating mixed liquor around the shaft's main primary downcomer-primary riser loop at a rate about eight times greater than the influent wastewater flow rate. Thus, influent wastewater is diluted on entering the shaft at an approximate ratio of 8:1. The resulting fluid velocities and detention times per pass are on the order of 1-1.5 m/sec (3-5 ft/sec) and 4-6 minutes, respectively.

The second major design change in the Eco II shaft is the relocation of the mixed liquor withdrawal point from the head tank to the R_2 secondary riser. R_2 runs within the R_1 primary riser from the withdrawal point to the surface. The entrance to R_2 is located slightly below the point at which influent wastewater and return sludge are injected into R_1. This arrangement precludes the possibility of short-circuiting untreated wastewater directly to the shaft effluent line. The two principal advantages of the revised method of transporting liquid from the shaft are: (1) the transfer of mixed liquor to the flotation tank with a higher dissolved gas content and (2) the provision of maximum single-pass treatment time following the initial contact of incoming substrate, biomass, and injected air.

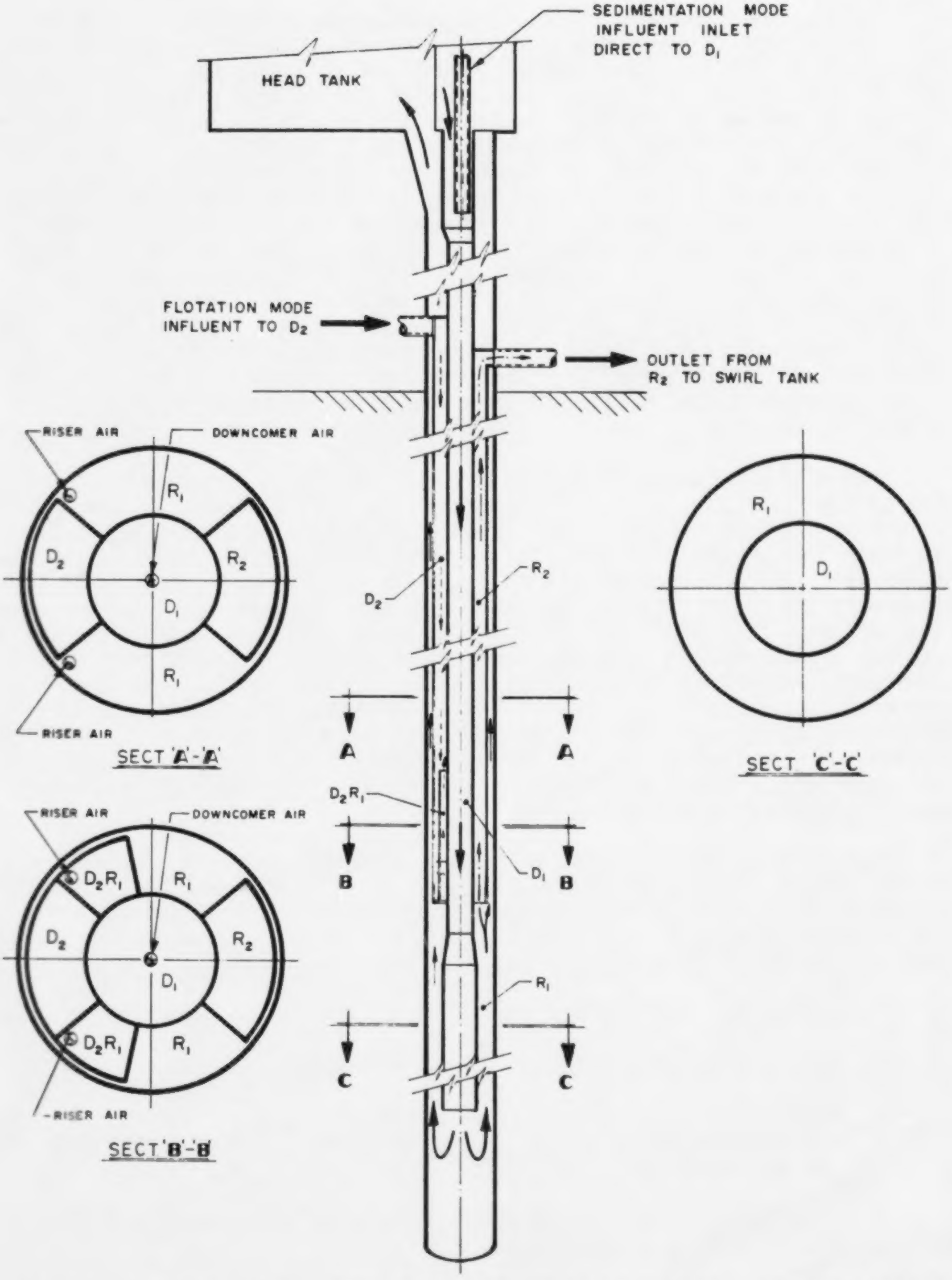
SEDIMENTATION MODE INFLUENT INLET DIRECT TO D_1
HEAD TANK
FLOTATION MODE INFLUENT TO D_2
OUTLET FROM R_2 TO SWIRL TANK
RISER AIR
DOWNCOMER AIR
R_1
D_2
R_2
D_1
RISER AIR
SECT 'A'-'A'
D_2
R_2
D_2R_1
D_1
R_1
A
A
B
B
C
C
RISER AIR
DOWNCOMER AIR
D_2R_1
R_1
D_2
R_2
D_1
D_2R_1
R_1
RISER AIR
SECT 'B'-'B'
R_1
D_1
SECT 'C'-'C'

FIGURE 3. ECO II SHAFT CONFIGURATION AND HYDRAULICS

Typical Eco I and Eco II gas voidage diagrams are presented in Figure 4 for a 133-m (435-ft) deep shaft with an assumed air injection depth in both legs of 53 m (175 ft). Gas voidage is defined as the volume of undissolved gas per unit volume of liquid. The shape of the voidage diagrams is directly related to the amount and distribution of air injected. In the Eco I design, approximately two-thirds of the total air supply is added to the downcomer and one-third to the riser. Conversely, the Eco II design adds only one-third to one-half of the total air supply to the primary downcomer and one-half to two-thirds to the primary riser through the D_2R_1 sections. More air is needed proportionally in the riser of an Eco II shaft due to the aforementioned introduction of wastewater and return sludge in the upper section of this leg. Another distinction is that total air requirements to sustain circulation at desired velocities are about 20 percent less for Eco II than Eco I. The smaller Eco II air requirements are due primarily to the more efficient air lift achieved by riser side air injection into the D_2R_1 sections rather than the larger R_1 leg and to a lesser degree to the higher velocities created by the channel configuration in the upper half of the shaft.

During shaft startup in either the Eco I or Eco II modes, air is initially fed to the riser leg only. The riser air displaces a portion of the liquid in the upper riser section. This produces a gas/liquid dispersion in the upper section of the riser with less net density than the liquid in the downcomer leg. The pressure differential thus created tends to push up on the liquid in the riser, causing the shaft contents to begin to circulate. The liquid pushed out the top of the riser into the head tank loses virtually all its gas bubbles by disengagement and venting before proceeding back down the downcomer, accounting for the absence of gas voidage in the upper section of the downcomer (Figure 4). Once stable circulation is achieved, the large difference in voidage betweeen the riser and downcomer sections is no longer needed and a portion of the air feed is transferred to the lower section of the downcomer. Only a small pressure differential is required to maintain circulation once it is established.

The shift in air injection toward the downcomer leg once circulation is established is greater in the Eco I than in the Eco II operating mode. The retention of a higher percentage gas voidage in the upper riser section of an Eco II shaft results in improved hydraulic stability and essentially eliminates the potential for hydraulic reversal to occur.

The claims made for the new Eco II bioreactor can be summarized as follows:

1. Anoxic zones within the shaft are eliminated.

2. Substrate removal kinetics and the shaft's overall biological profile are enhanced owing to the better matching of oxygen demand with oxygen availability.

3. Single-pass plug flow treatment time in a high-intensity aerobic environment is maximized.

4. Total air requirements are decreased and organic loading capacity is increased by 15-20 percent each.

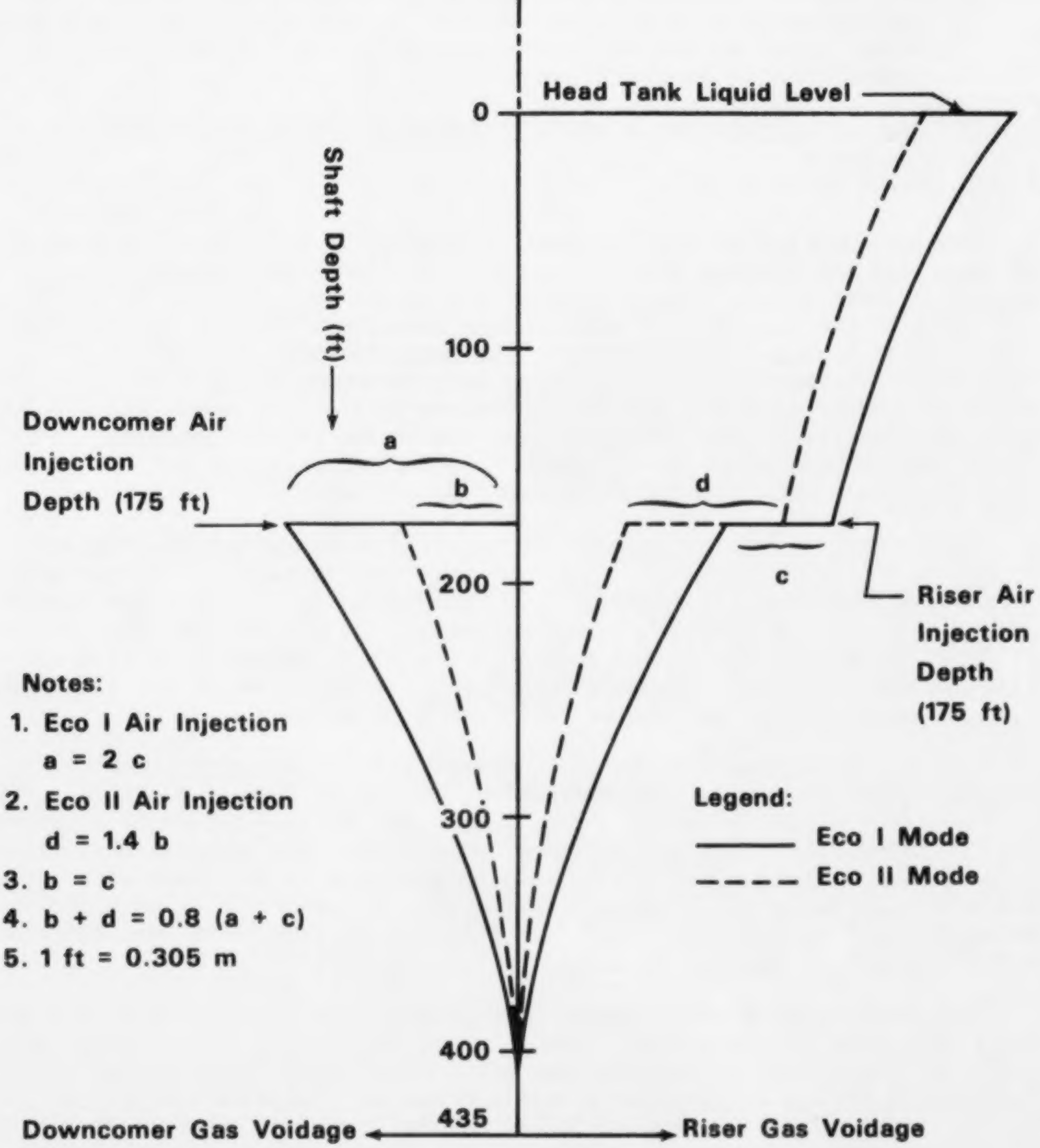
Head Tank Liquid Level
0
100
200
300
400
435
Shaft Depth (ft)
Downcomer Air
Injection
Depth (175 ft)
a
b
d
c
Riser Air
Injection
Depth
(175 ft)
Notes:
1. Eco I Air Injection
a = 2 c
2. Eco II Air Injection
d = 1.4 b
3. b = c
4. b + d = 0.8 (a + c)
5. 1 ft = 0.305 m
Legend:
Eco I Mode
Eco II Mode
Downcomer Gas Voidage
Riser Gas Voidage

FIGURE 4. TYPICAL ECO I AND ECO II GAS VOIDAGE DIAGRAMS

5. Dissolved gas drive for suspended solids separation by flotation is optimized.

6. The overall improvement effected in flotation performance by Item 5 above reduces or eliminates the need for the addition of polymers or other flotation aids to achieve a high degree of suspended solids removal in the final effluent.

7. Hydraulic stability of the circulating mixed liquor is improved.

ECO II SYSTEM DESCRIPTION

The complete Eco II system, shown schematically in Figure 5, is comprised of more than a bioreactor and a flotation clarifier. Auxiliary tanks and piping networks are an integral part of system design. Figure 5 is representative of the Deep Shaft flotation mode demonstration facility recently installed at Ithaca. For simplicity, the shaft has been depicted as a single tube with a divider wall forming the primary downcomer and primary riser legs. The D_2 secondary downcomer and the R_2 secondary riser are shown, but the short D_2R_1 secondary riser sections have been omitted for clarity purposes. The operational scheme of an Eco II system is described below, as taken primarily from the Ithaca Operating and Maintenance Manual (8).

In contrast to an Eco I head tank, which is enclosed but supplied with a direct vent to the atmosphere, an Eco II head tank is enclosed and operates at a supra atmospheric pressure of 2000-5600 kgf/m^2 (3-8 psig). Oxygen dissolution in the head tank is thereby minimized, allowing higher concentrations of DO to be carried over to the downcomer than would remain in equilibrium with the atmosphere. Disengaged bubbles exit the head tank as off gas to the foam oxidation tank. The foam oxidation tank is vented.

Eco has determined that foam can be more efficiently stabilized in a separate sidestream tank. The deleterious effects of foam on microbial activity are thus minimized in the shaft, permitting the utilization of a somewhat smaller bioreactor volume and a higher F/M loading. Eco's operational experiences to date indicate that little foam is generated in the shaft with typical municipal wastewaters. Sidestream foam oxidation appears to be more pertinent to naturally foaming wastes found in the discharges from breweries, dairy products processing, and textile manufacturing (9).

Partially treated or collapsed foam is displaced to the holding tank and eventually back into the shaft. Raw influent and floated return sludge also enter the shaft from the holding tank after first being contacted and mixed in a trough that runs along and slightly below the flotation tank beach. Bottom return sludge is recycled to the holding tank in a separate line. The mixture of influent wastewater, float return sludge, bottom return sludge, and collapsed foam is air lifted into the shaft via the secondary downcomer (D_2) and secondary risers (D_2R_1), as illustrated previously in Figure 3.

The swirl tank briefly intercepts mixed liquor as it is transported to the flotation tank. The residence time of 10-15 seconds provided by the swirl tank (at average flow) is long enough to strip out large bubbles that

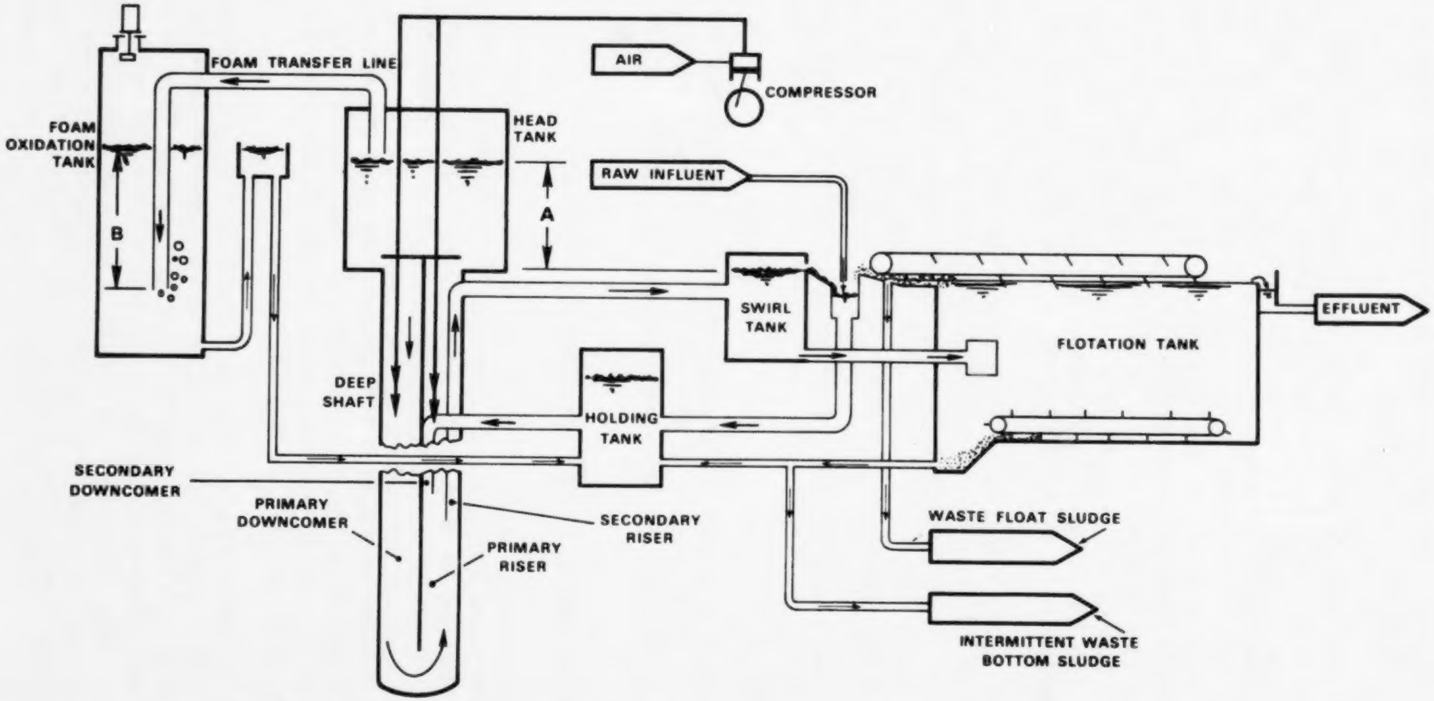
FOAM TRANSFER LINE
FOAM OXIDATION TANK
B
HEAD TANK
A
AIR
COMPRESSOR
RAW INFLUENT
SWIRL TANK
FLOTATION TANK
EFFLUENT
DEEP SHAFT
HOLDING TANK
SECONDARY DOWNCOMER
PRIMARY DOWNCOMER
PRIMARY RISER
SECONDARY RISER
WASTE FLOAT SLUDGE
INTERMITTENT WASTE BOTTOM SLUDGE

FIGURE 5. SCHEMATIC OF DEEP SHAFT SYSTEM USING FLOATATION CLARIFICATION

would disrupt flotation, but not so long to promote effervesence of dissolved gases and/or small bubbles.

The pressure required to force the mixed liquor out through the R_2 secondary riser is provided by the difference between the liquid levels in the head tank and the swirl tank (distance A in Figure 5). This head differential offsets the friction losses in the R_2 section. Therefore, as the liquid level in the head tank increases, the output of R_2 increases. In order to limit the height requirements of the head tank, it is pressurized as previously indicated to between 2500 and 5600 kgf/m^2 (3 and 8 psig). This is equivalent to increasing the liquid level in the head tank. The exact pressure in the head tank is governed by the liquid level in the foam oxidation tank (distance B in Figure 5). The height of liquid above the bottom of the air outlet pipe in the foam oxidation tank is proportional to the pressure in the head tank.

If polymer or other chemicals are required to aid in flotation separation, they are introduced to the mixed liquor in the pipe directly beneath the swirl tank feeding the flotation tank. Variable rate skimmers are provided to collect the floated solids and draw them toward and onto the beach. Some fraction of the mixed liquor solids (mainly the inerts) will not be amenable to flotation and will sink instead. These solids are collected by bottom scrapers and a screw conveyor for direct gravity return to the holding tank, as shown in Figure 5, or alternately for pumped return to the influent trough fronting the beach.

Wasting of floated solids is accomplished through an orifice in the beach. An automatic timed valve in the waste sludge line can be used to control the float solids wasting rate. Bottom solids wasting is controlled in a similar manner.

FLOTATION DESIGN AND OPERATIONAL IMPROVEMENTS - ECO III

Based on experimental work at their Paris, Ontario pilot facility, Eco claims to have advanced the state-of-the-art of flotation clarification, particularly as applied to Deep Shaft technology. The advancements made are reportedly in the areas of eliminating sophisticated (but operationally troublesome) instrumentation and controls, improving performance reliability during periods of large hydraulic fluctuations through the system, and enhancing hydraulic and mass loading capacities. Eco has not disclosed the details of the Eco III design improvements to EPA at the time of this writing. Eco will publish these details at a future date.

The first Deep Shaft system that will include the Eco III concept is currently under construciton for the City of Portage la Prairie, Manitoba, Canada (9). This plant is designed to treat an average flow of 14,300 m^3/day (3.75 mgd) with wet weather capacity to 37,800 m^3/day (10 mgd). The Ithaca demonstration unit may eventually be equipped with an Eco III flotation tank. In effect, future Eco III Deep Shaft plants will really consist of a third-generation bioreactor and a second-generation flotation tank. For convenience, this flowsheet will be referred to simply as Eco III in the remainder of the paper.

THE ITHACA DEMONSTRATION PROJECT

SITE SELECTION

The City of Ithaca is located in the state of New York at the south end of Cayuga Lake approximately 80 km (50 mi) southwest of Syracuse, New York. The climate in this northern United States region ranges from severe freezing in the winter to high temperatures in the summer. Ithaca is the home of Cornell University, which contributes substantially to the City's waste load.

Ithaca's existing wastewater treatment plant provides primary clarification and secondary treatment using the trickling filter process for a population equivalent of 40,000. The primary treatment section of the plant dates back to 1938, and the trickling filter units were completed in 1961. These facilities have an average design flow rate of 15,100 m^3/day (4 mgd) with peak flow capacity to 60,600 m^3/day (16 mgd). Currently, the average dry weather flow is approaching 26,500 m^3/day (7 mgd). Peak flows in excess of 60,600 m^3/day (16 mgd) occur occasionally throughout the year.

The City's treatment plant is obviously overloaded and needs to be expanded and upgraded to provide satisfactory secondary treatment in line with Federal and State effluent regulations. The plant property borders greenbelt park areas that have been developed along the Cayuga Lake shoreline. The presence of a major university and the tourist and recreational appeal of the lake have produced an environmentally active and aware local constituency. The Ithaca citizenry is consequently opposed to any treatment plant expansion proposal that would encroach on the adjacent parkland.

The above dilemma faced by Ithaca was a paramount incentive in the City's decision to seek an EPA grant to evaluate and demonstrate the Deep Shaft Process. City officials are anxious to identify potential new wastewater treatment alternatives that minimize land utilization.

In addition to the assured continuing interest and cooperation on the part of the City due to the above situation, EPA selected Ithaca as the site of the first municipal assessment of Deep Shaft technology in the United States for the following reasons:

1. The City's consulting firm is currently preparing a Facilities Plan, which is the first step in the approval process for an EPA treatment plant construction grant. The Deep Shaft evaluation can be completed in time to consider the process for the Ithaca plant expansion prior to final design selection. Plant design represents Step 2 in the U.S. Construction Grants Program sequence.

2. The existing wastewater flow is sufficiently large to permit a sidestream demonstration study without disrupting main plant operations.

3. A diversity of influent waste streams including raw wastewater, primary effluent, trickling filter effluent, and anaerobic digester supernatant are available for feed to the Deep Shaft pilot unit.

4. The Ithaca wastewater treatment plant is staffed with well-qualified operating and supervisory personnel. The City also has an excellent relationship with the engineering firm that will oversee the conduct of the project and evaluate the process data and cost effectiveness.

It is believed the above factors will all contribute to the successful completion of a comprehensive pilot demonstration project. The results and engineering assessments to be derived therefrom should have nationwide impact in defining the future applicability of Deep Shaft technology to the wastewater treatment problems of the United States.

DEMONSTRATION SYSTEM DESCRIPTION AND DESIGN

The Ithaca pilot demonstration system was flexibly designed to permit operation in two different modes. These two modes are flotation clarification utilizing an Eco II flowsheet (Figure 5) and gravity clarification preceded by vacuum degassing utilizing the original ICI flowsheet (Figure 2). The shaft has been constructed to operate with secondary downcomer and riser sections (Eco II) or without them (ICI). A single liquid/solids separation tank has been provided to serve as both a flotation clarifier and a gravity clarifier.

The steel shaft casing has an inside diameter of 44 cm (17.25 in.), a depth of 136 m (446), and is grouted to the geological formation with cement (10). The inner concentric primary downcomer has an outside diameter of 30 cm (11.75 in.) and a depth of 133 m (436 ft). The annulus formed between the outer casing and the inner primary downcomer constitutes the primary riser. Excluding the head tank, the volume of Ithaca's shaft is 20.5 m^3 (5418 gal).

The secondary downcomer (D_2) begins about 3 m (10 ft) below the base of the head tank and runs within the primary riser (R_1) to a depth of 61 m (200 ft). The short D_2R_1 secondary riser sections, which are connected to the D_2 downcomer, discharge influent wastewater and return sludge into the R_1 primary riser about 9 m (30 ft) up from the bottom of the D_2 section. The opening to the secondary riser mixed liquor withdrawal pipe (R_2) is also positioned at the 61-m (200-ft) level within the primary riser. Air is injected at two locations: 55 and 60 m (180 and 196 ft) down from the base of the head tank in the primary downcomer (D_1) and secondary risers (D_2R_1), respectively (8). Referral to Figure 3 will assist the reader in comprehending Ithaca pilot shaft internal geometry.

The dimensions of the liquid/solids separation unit are 3.4-m (11.25-ft) wide x 11.0-m (36-ft) long x 4.0-m (13-ft) SWD (9). The surface area equals 37.6 m^2 (405 ft^2). This tank is equipped with both top and bottom variable speed scrapers to collect floated solids (or scum during sedimentation operation) and settled solids, respectively. By opening and closing a few valves, the tank is readily converted from the flotation mode to the gravity clarification mode and vice versa.

The vacuum degasser shown preceding the sedimentation tank in Figure 2 consists of 16.5-m (54-ft) high tower with an inner concentric pipe (10).

Mixed liquor from the surge tank flows up the annulus under a vacuum of approximately 10^{-6} atmospheres. Effluent from the degassing tower is piped directly into the sedimentation tank (10).

The vacuum degasser and surge tank are employed only when the pilot system is operated in the gravity clarification mode. Since it would be counterproductive to maximize the gas content of the mixed liquor leaving the shaft in the gravity clarification mode, the secondary downcomer and riser sections are not utilized and the head tank is not pressurized. In addition, the foam oxidation, swirl, and holding tanks and the influent trough used in the flotation clarification mode are idle during gravity clarification operations. Influent wastewater combined with underflow return sludge enters the head tank directly. Mixed liquor destined for gravity clarification is displaced from the head box into the top of the surge tank.

Air is supplied to the shaft from a 15-kW (20-hp) compressor. Startup operations (discussed later on) have determined that a constant air supply rate of 25 l/sec (53 scfm) is required for maintenance of desired circulation velocities in the Eco II shaft configuration (flotation clarification mode). This air supply rate is equivalent to an oxygen supply rate of 650 kg/day (1434 lb/day). The above air supply is distributed 15.6 l/sec (33 scfm) to the riser side via the D_2R_1 secondary risers and 9.4 l/sec (20 scfm) to the D_1 primary downcomer. At this air supply rate, the compressor operates at a line draw of roughly 11 wire kW (15 wire hp). Air supply requirements and distribution for the ICI or Eco I shaft configuration (gravity clarification mode) have not yet been defined. However, it is estimated total air supply requirements will be 15-20 percent greater with approximately two-thirds delivered to the downcomer and one-third to the riser.

The relationship of the Deep Shaft pilot demonstration plant to the Ithaca main plant treatment facilities is illustrated schematically in Figure 6. A sidestream of raw wastewater is normally diverted to the Deep Shaft pilot unit after screening and grit removal. A line was also recently installed allowing a sidestream of primary effluent to be fed directly to the demonstration system, when desired. No piping has been provided to introduce trickling filter effluent to the Deep Shaft plant at this time. An evaluation using trickling filter effluent as feed would be on interest only in the event nitrification became a requirement. Digester supernatant can enter the Deep Shaft facilities via its recycle to the main plant headworks. Treated effluent and waste sludge from the demonstration plant are pumped to the influent end of the main plant primary clarifiers to preclude any discharge of pilot effluent streams to the receiving water. To facilitate winter operation and protect the solids separation unit during flotation operations, a prefabricated 15-m x 12-m (50-ft x 40-ft) metal building has been provided to house the entire Deep Shaft demonstration facility.

Ithaca's Deep Shaft demonstration plant was designed to have a nominal capacity in the flotation clarification mode (Eco II flowsheet) of 757 m^3/day (200,000 gpd) at an influent BOD concentration of 150 mg/l. Hydraulically, the system can handle 1325 m^3/day (350,000 gpd). Due to solids loading constraints and corresponding lower attainable MLSS concentrations, nominal design capacity in the gravity clarification mode (ICI flowsheet) is only 379 m^3/day (100,000 gpd). Diurnal flow variation capability is available for

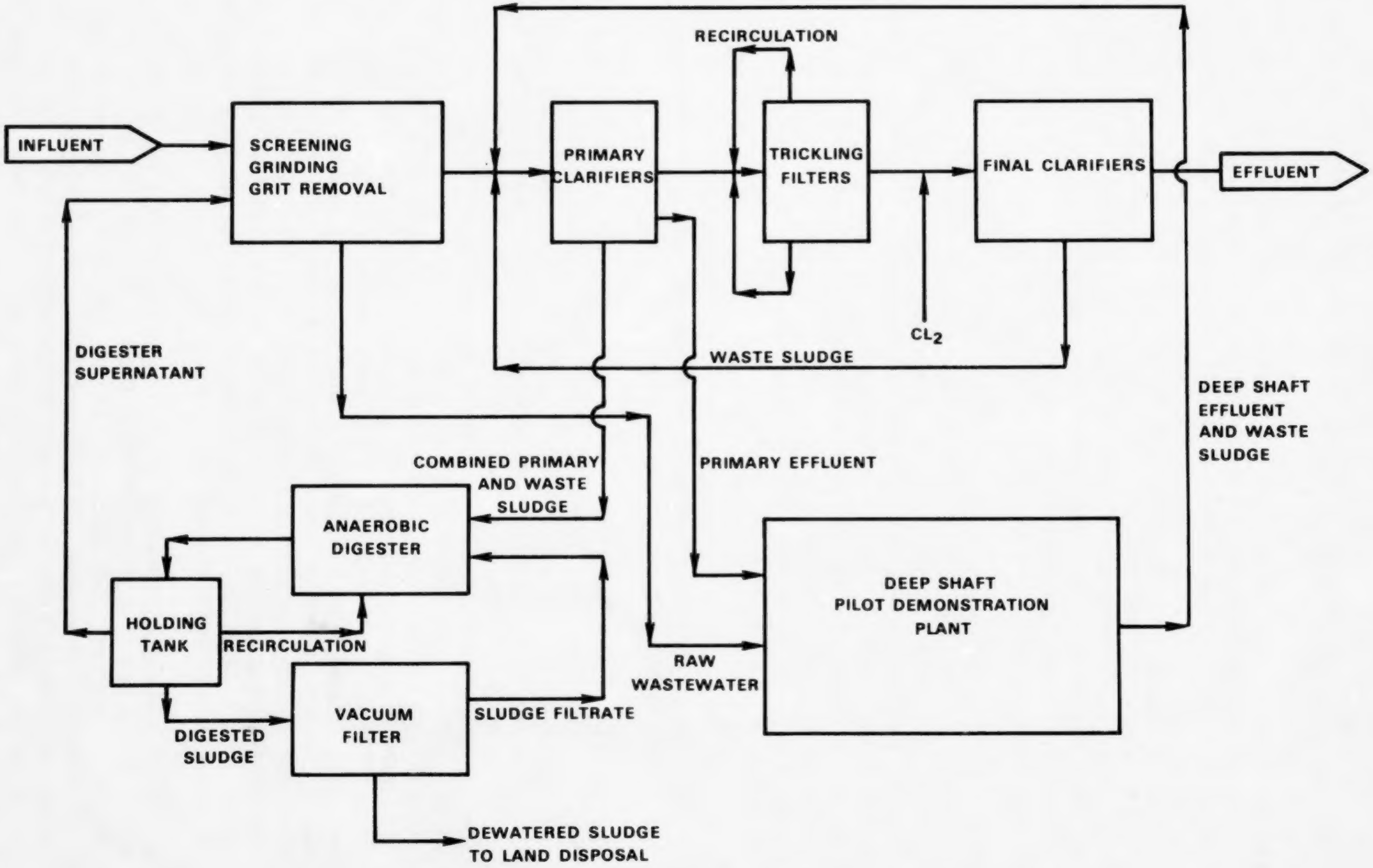
INFLUENT
SCREENING GRINDING GRIT REMOVAL
PRIMARY CLARIFIERS
RECIRCULATION
TRICKLING FILTERS
FINAL CLARIFIERS
EFFLUENT
CL2
WASTE SLUDGE
DIGESTER SUPERNATANT
DEEP SHAFT EFFLUENT AND WASTE SLUDGE
COMBINED PRIMARY AND WASTE SLUDGE
PRIMARY EFFLUENT
ANAEROBIC DIGESTER
DEEP SHAFT PILOT DEMONSTRATION PLANT
HOLDING TANK
RECIRCULATION
RAW WASTEWATER
VACUUM FILTER
SLUDGE FILTRATE
DIGESTED SLUDGE
DEWATERED SLUDGE TO LAND DISPOSAL

FIGURE 6. FLOW DIAGRAM OF ITHACA'S EXISTING WASTEWATER TREATMENT PLANT

programmed stressing. A complete listing of design criteria for the two operating modes is summarized in Table 1.

PROJECT OBJECTIVES

Eco-Research Ltd. would have preferred to limit the Ithaca Deep Shaft evaluation to flotation clarification only. EPA's position, however, was that the original ICI gravity clarification flowsheet, which is still marketed in the U.K., Europe, and Japan, should be included in the study to add perspective. A compromise was effected wherein both operating modes could be assessed, but by designing flexibility into the system, dual units were avoided. With this brief introduction, project objectives are given below (12).

1. To evaluate the efficiency and reliability of the Deep Shaft Process in treating a typical municipal wastewater under varied operating conditions.

2. To provide guidance to U.S. engineers on the applicability of the Deep Shaft Process to U.S. municipal treatment practices and to define critical design and operating parameters.

3. To compare solids separation performance and associated shaft operating characteristics in both the flotation and gravity clarification modes.

4. To develop capital and operating cost estimates to apply Deep Shaft technology to a wide range of municipal projects.

5. To define flotation chemical aid requirements, if any.

6. To compare Deep Shaft operation and performance (including sludge production and energy consumption) to traditionally accepted operating and performance values of conventional activated sludge systems, both air and oxygen.

7. To evaluate process performance with and without alum addition for phosphorus removal.

8. To compare process performance on raw wastewater feed vs. primary effluent feed.

9. To determine whether typical U.S. municipal treatment plant personnel can be trained to effectively, routinely, and independently operate a Deep Shaft Process.

SYSTEM STARTUP SUMMARY

The startup program was commenced on October 8, 1979, and continued until November 12, 1979, under Eco personnel. The demonstration plant was then turned over to the City who operated it through November 30, 1979, at which time the system was shut down for flotation tank modifications and belated installation of the primary effluent feed line. Following the

TABLE 1. DESIGN CRITERIA FOR ITHACA DEEP SHAFT DEMONSTRATION SYSTEM

Parameter	Flotation Clarification Mode	Gravity Clarification Mode
Bioreactor		
Nominal design flow, m^3/day	757	379
mgd	0.2	0.1
MLSS, mg/l	10,000	5000
SRT[a], days	2.1	2.1
F/M loading[b], kg BOD$_5$/day/kg MLVSS	0.74	0.74
Volumetric organic loading[c],		
kg BOD$_5$/day/m^3	5.54	2.77
lb BOD$_5$/day/1000 ft^3	346	173
Detention time[d], minutes		
nominal (Q)	39	78
actual (Q + R)	24	39
Solids Separation Unit		
Surface overflow rate,		
m^3/day/m^2	20.1	10.1
gpd/ft^2	494	247
Mass loading[e], kg TSS/day/m^2	321	103
lb TSS/day/ft^2	66	21
Return sludge flow rate, % of Q		
float recycle	20	N.A.
bottom recycle	40	100
Return sludge concentration, % TSS		
float solids	7-10	N.A.
bottom solids (intermittent)	3-4	1-2
Waste activated sludge[f],		
kg TSS/kg BOD$_5$ removed	0.75	0.75
kg VSS/kg BOD$_5$ removed	0.45	0.45
kg TSS/day	77	39
lb TSS/day	170	85

(continued on next page)

TABLE 1. (continued)

[a] Defined as kg MLSS in bioreactor/(kg TSS lost in waste activated sludge and final effluent/day).

[b] F/M loading assumes an influent BOD_5 concentration of 150 mg/l and a mixed liquor volatile content of 75 percent.

[c] Volumetric organic loading estimated assuming influent BOD_5 of 150 mg/l, a mixed liquor volatile content of 75 percent, a nominal shaft diameter of 0.44 m (17.25 in.), and a shaft depth of 136 m (446 ft).

[d] Actual detention time based on sludge recycle rates of 100 percent in the gravity clarification mode and 20 percent and 40 percent, respectively, for float solids and bottom solids in the flotation clarification mode.

[e] Mass loadings based on total sludge return rates of 60 and 100 percent and MLSS concentrations of 10,000 and 5000 mg/l, respectively, in the flotation and gravity clarification modes.

[f] Activated sludge wasted per unit of BOD_5 removed based on an influent BOD_5 of 150 mg/l and an effluent BOD_5 of 15 mg/l.

departure of the Eco process startup engineer, it soon became evident that the City operators still lacked sufficient judgment to maintain the proper wasting balance between float solids and bottom solids. Deteriorating performance was soon evident in increasing effluent suspended solids. The modifications prescribed by Eco to correct the problems were to provide separate control over flotation tank internal fluid recirculation and sinking sludge return rate.

The 53 days (36 Eco, 17 City) of operation documented in Eco's startup report (11) were comprised of 3 days at 70 percent of design flow, 46 days at the flotation mode design flow of 757 m^3/day (200,000 gpd), 2 days at 150 percent of design flow, and 2 days at reduced flow to repair equipment. Operating results for the 36 days Eco was in charge of the startup program are summarized below (11):

1. Effluent total BOD_5 averaged 15 mg/l vs. 135 mg/l entering for a removal of 89 percent. Total BOD_5 results are plotted in Figure 7. The EPA 30 mg/l minimum acceptable effluent BOD_5 limitation is shown for reference. Effluent soluble BOD_5 averaged 6 mg/l during the same period.

2. Suspended solids removal averaged 91 percent with 30 mg/l in the effluent against 334 mg/l in the influent, which exactly meets EPA's minimum acceptable suspended solids limitation. These results were achieved, with the exception of 3 trial days, without the use of a polymeric flocculation aid. Suspended solids data are presented graphically in Figure 8. As indicated, effluent suspended solids did

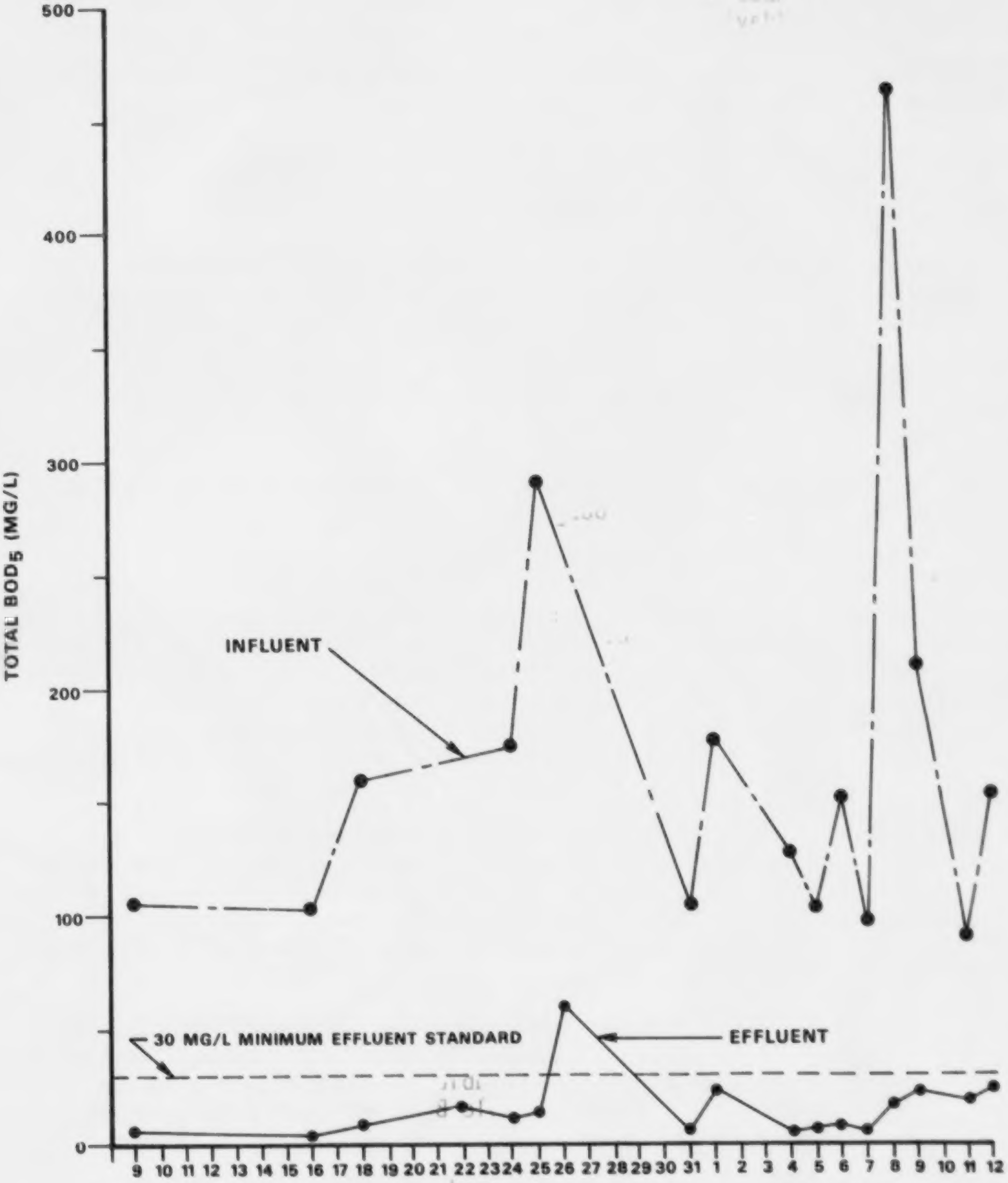

FIGURE 7. ITHACA DEEP SHAFT BOD_5 DATA DURING STARTUP

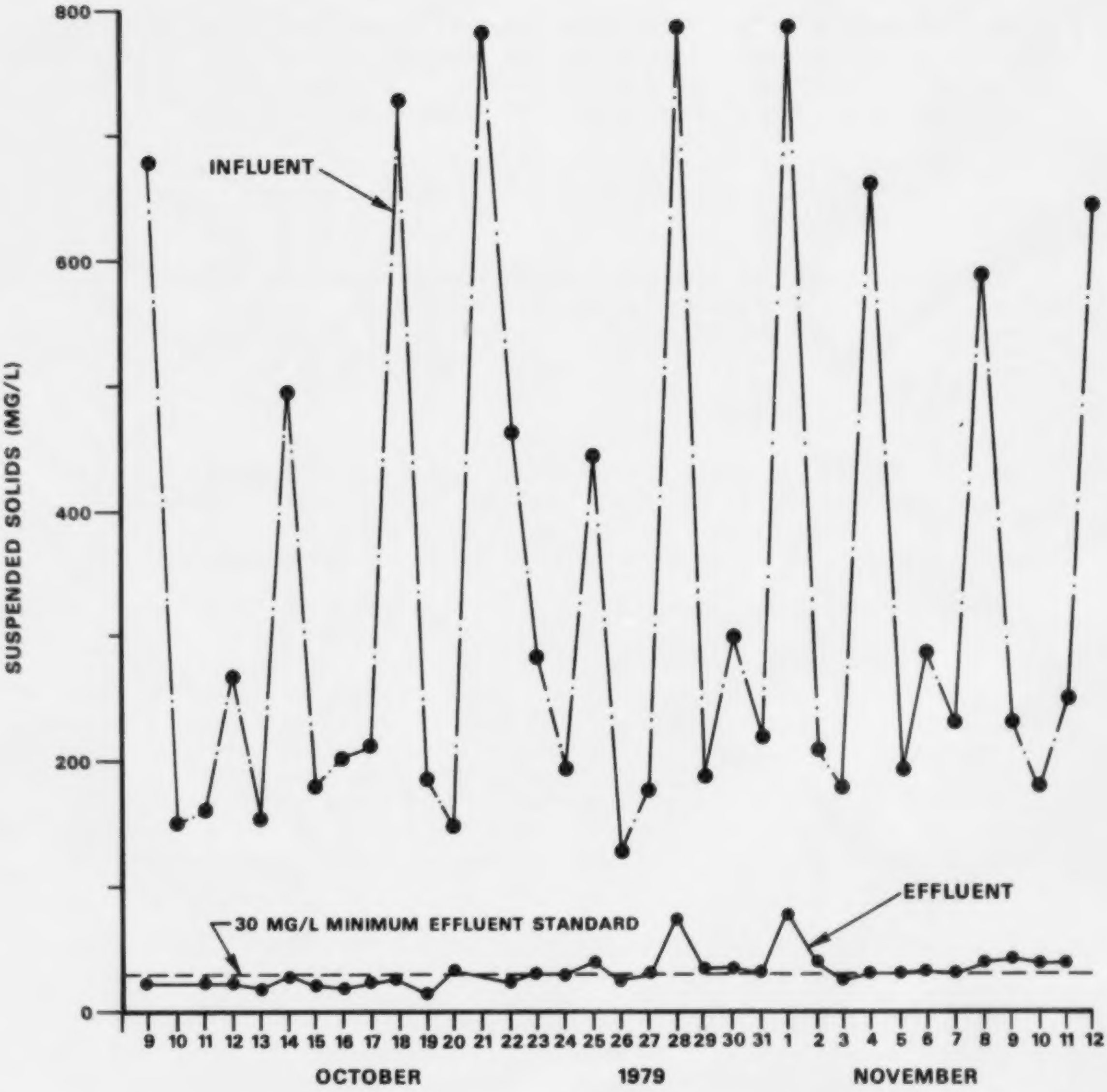

FIGURE 8. ITHACA DEEP SHAFT SUSPENDED SOLIDS DATA DURING STARTUP

exceed 30 mg/l on occasion.

3. An average effluent total COD concentration of 66 mg/l was achieved. Based on an average influent total COD of 485 mg/l, removal was 86 percent for the period.

4. Effluent soluble COD mirrored the acclimatization of the Deep Shaft biomass, dropping from an initial level of 100-110 mg/l to 50-60 mg/l after 2-1/2 weeks to 20-30 mg/l after 3-1/2 weeks. Influent total COD values are plotted against effluent soluble COD values in Figure 9.

5. Suspended solids in the float blanket (top return) generally ranged from 8-8.5 percent; underflow solids (bottom return) ranged from 2.5-3.5 percent.

6. Due partially to anaerobic digester supernatant solids returned to the main plant headworks, the reactor biomass developed rapidly reaching an operating MLSS concentration of 10,000 mg/l within 2 days. By the second day, effluent specifications of 30 mg/l BOD_5 and 30 mg/l suspended solids were being produced at 70 percent of design flow (530 m^3/day = 140,000 gpd).

7. Except for rotifers, a diversified microfauna was observed microscopically within 6 days of startup. (Note: Rotifers, which are typically present in mature conventional activated sludges, have never been detected in Deep Shaft municipal wastewater cultures.)

The aforementioned recycle of digester supernatant to the headworks of the Ithaca plant caused problems for operating personnel during startup. The anaerobic digester's mechanical agitator is currently non-operational. To provide needed stirring in the digester, overflow digester liquor is continuously pumped to a large holding tank and back to the digester (see Figure 6). Twice a week (usually Sunday and Thursday nights), the pump is turned off, solids are allowed to settle in the holding tank, and supernatant is diverted from the tank to the plant's grit chamber. These digester supernatant "dumps" last 5-6 hr, and despite the preceding settling period in the holding tank contribute high slug loadings of organics and suspended solids to the Ithaca raw wastewater.

The demonstration plant raw wastewater sidestream takeoff is located near the exit from the grit chamber. Consequently, shaft bioreactor influent received the effect of the "dumps" virtually undampened. Influent total COD and suspended solids concentrations increased 10-20 fold during the 5- to 6-hr "dump" periods. The solids were highly mineralized (50 percent $\bar{+}$ volatile fraction) and, therefore, did not increase influent total BOD_5 by the same proportion. This shock condition made it impossible to hold system biomass inventory at prescribed levels, often increasing MLSS to 15,000 mg/l and above. The digester supernatant was also observed to have a temporary inhibitory impact on mixed liquor respiration rates.

The biweekly digester supernatant "dump" policy produced the following effects on Deep Shaft performance. Effluent suspended solids increased from

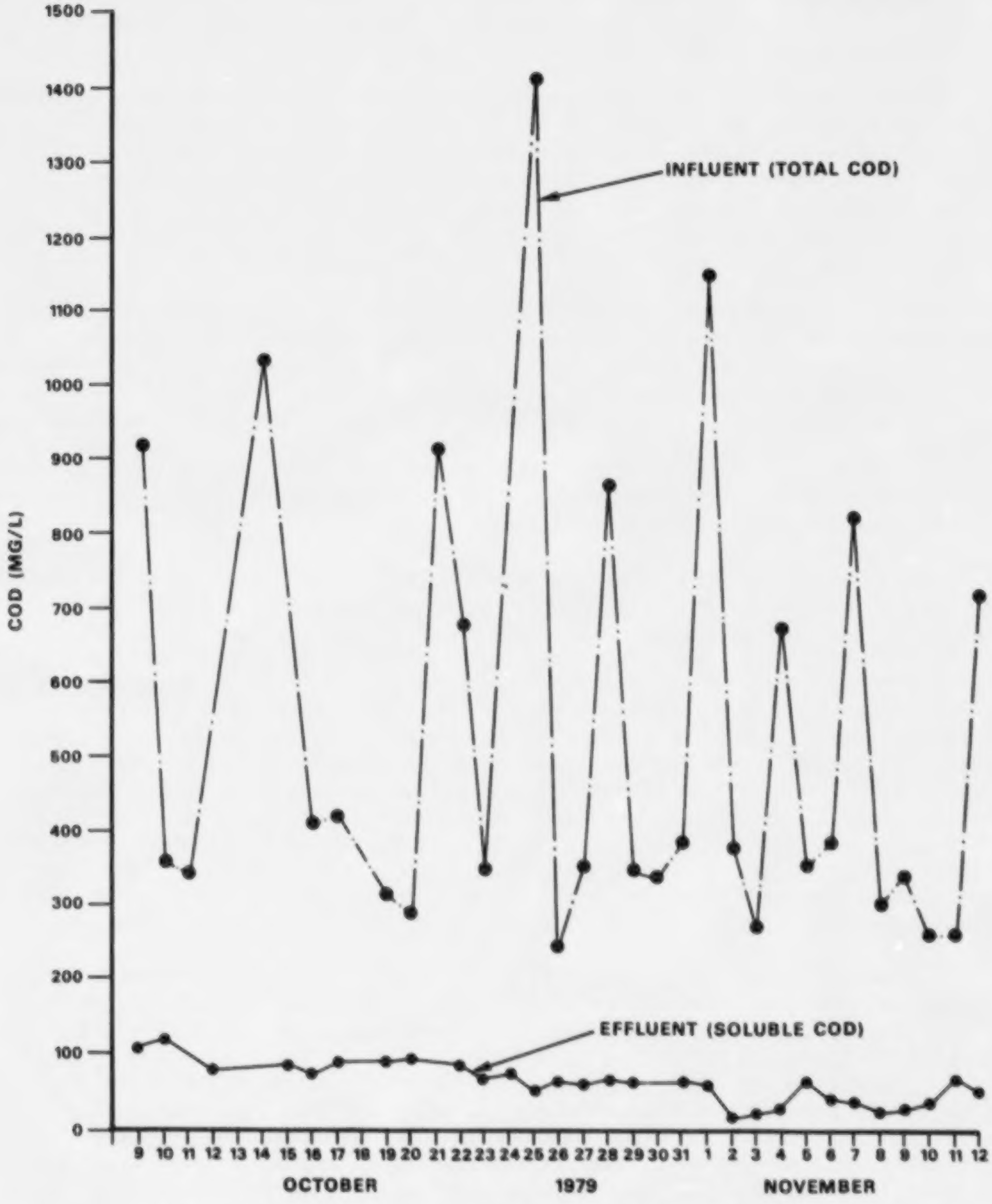

FIGURE 9. ITHACA DEEP SHAFT COD DATA DURING STARTUP

a typical background level of 30 mg/l ± to peaks of 40-80 mg/l. Effluent total COD exhibited corresponding increases, peaking at concentrations ranging from 100-200 mg/l. Recovery to steady-state effluent quality generally occurred within 12 hr of the initiation of the shock loadings. Recovery time could be minimized by careful operation and anticipation of the "dumps."

Float quality was also impaired by the shock loadings, thinning out from a normal thickness of about 60 cm (24 in.) to 15 cm (6 in.) or less. A 3-mg/l dose of cationic polymer (Stockhausen "Praestol" 423K) was beneficial in helping to maintain desirable float characteristics during the "dumps," but at the expense of increased effluent suspended solids, probably due in Eco's opinion to rendering more flocs neutrally buoyant (11). The float blanket also typically recovered to "pre-dump" stability within 12 hr.

The difficulties that could be anticipated over a long-term evaluation period from the digester supernatant slugs became more evident when the demonstration plant was turned over to the City. For the 17 days from November 13 through November 30, 1979, with City operators in control, effluent quality deteriorated on the average to 27 mg/l total BOD_5 (69 percent removal), 106 mg/l suspended solids (70 percent removal), and 151 mg/l total COD (65 percent removal). Effluent deterioration was attributed to the lack of operator experience in adjusting process sludge wasting requirements, particularly in response to the supernatant discharges.

The primary effluent feed line was initially thought of as an optional wastewater source to be utilized only during planned experimental phases. Following the startup experiences, a joint decision was made to switch the shaft influent source from degritted raw wastewater to primary effluent during the 5-6 hr that future supernatant "dumps" occur. By taking advantage of the solids removal capacity of the main plant primary clarifiers, shaft influent pollutant levels should not increase so dramatically and rapidly. Although the pilot system exhibited excellent recovery capability, it was felt that coping with frequent supernatant discharges would dilute the major emphases of the project and disrupt the accurate recording and analysis of critical data such as excess sludge production and flotation tank response to routine variations in loading. In the impending Ithaca plant expansion, the City's engineering design firm will in all likelihood modify the existing sludge handling scheme and smooth out internal recycle flows or treat them separately.

Installation of the primary effluent feed line and the aforementioned flotation tank modifications proceeded from December 1, 1979, to February 6, 1980. The demonstration plant was then restarted under Eco supervision. The Deep Shaft system is scheduled to be turned over to the City again by April 1, 1980. Operating and supervisory roles within the Ithaca plant staff in relation to the pilot plant have been more clearly defined since the initial startup. This should lead to improved process understanding and operational judgment. One of the more important measuring sticks of process practicality, however, will be the degree of ease or difficulty City personnel experience in establishing routine operating and maintenance schedules and consistent performance. Their progress will be closely monitored in the coming months as they attempt to carry out the evaluation program described in the following subsection under the overall engineering control of the City's consultant, who is also relatively inexperienced regarding Deep Shaft technology, and with minimal dependence on Eco.

EXPERIMENTAL DESIGN

Evaluation Program

Once the Ithaca staff has achieved a stable operating regime, a 64-week evaluation program will be undertaken. The program tentatively consists of 42 weeks of phased evaluations in the flotation clarification mode (Eco II flowsheet, Figure 5) and 22 weeks in the gravity clarification mode (ICI flowsheet, Figure 2). Future incorporation of Eco III concepts in the Ithaca flotation tank could alter or expand the evaluation schedule. The evaluation program, outlined in Table 2, is designed to address the objectives listed earlier in this section, principally those related to process performance, operational stability, operating criteria, and process costs.

Analytical Program

The large analytical load to be generated by the aforegoing evaluation program will be shared between Ithaca's treatment plant laboratory and a commercial laboratory (12). The City will conduct total and volatile suspended solids analyses; in-plant DO, pH, and wastewater temperature determinations; and 1-liter sludge settling tests. The analyses to be assigned to the commercial laboratory consist of total and soluble BOD_5, total and soluble COD, the nitrogen series, total phosphorus, fecal coliforms, oil and grease, chlorides, alkalinity, and heavy metals. In addition, Ithaca operators will be responsible for monitoring flow rates (including waste activated sludge), power consumption, and mechanical components as well as performing regular visual checks.

The streams to be sampled include system influent, system effluent, mixed liquor, float recycle and waste, and bottom recycle and waste. Major contaminants of interest (BOD_5, COD, suspended solids) as well as DO, pH, temperature, and sludge settling will be assayed 5 days per week. Nutrients will only be analyzed once per week, except for special sampling periods when they will be determined 5 days per week. Heavy metals, chlorides, alkalinity, oil and grease, and fecal coliforms will be run periodically. During certain designated test periods, effluent soluble BOD_5 frequency will be increased to trace process response to rapidly changing organic load patterns.

Project Costs and Staffing

There are four parties to the Ithaca demonstration grant project: the EPA Office of Research and Development as the grantor, the City of Ithaca as the grantee, Eco-Research Ltd. as proprietary process subcontractor to the City, and the consulting firm of Sterns and Wheler (S&W), Cazenovia, New York, as engineering subcontractor to the City. The Principal Investigator for the project is an employee of S&W. The responsibilities of S&W include supervising demonstration plant operations and data collection, evaluating Deep Shaft process performance and costs, convening meetings to discuss

TABLE 2. TENTATIVE ITHACA EVALUATION PROGRAM

Phase	No. of Weeks
Flotation mode	
1. Constant flow - raw wastewater feed	6
2. Diurnal flow with periodic wet weather peaking corresponding to plant wet weather flow - raw wastewater feed	12
3. Combined organic and hydraulic overloads by increasing flow incrementally at normal strength maintaining a 1.5/1.0 diurnal peaking factor - raw wastewater feed	8
4. Primary effluent feed - diurnal flow	8
5. Alum addition on primary effluent feed - diurnal flow	4
6. Alum addition on raw wastewater feed - diurnal flow	4
Subtotal	42
Gravity mode	
1. Constant flow - raw wastewater feed	6
2. Diurnal flow - same as Step 2 above	12
3. Combined organic and hydraulic overloads - same as Step 3 above	4
Subtotal	22
Grand Total	64

progress and alter direction (if required), and preparing progress reports and a final report for publication. Eco's responsibilities to date have consisted of turnkey procurement, construction, and commissioning of the demonstration plant and training Ithaca and S&W personnel. They will also provide process expertise and advice, as needed, throughout the project.

Eco's contractual responsibility to the City has been primarily tied to the front end of the project. At the point the pilot system is turned over to the City around April 1, 1980, responsibility for the remainder of the project will shift to S&W. EPA considers this separation of vested interests and independent, unbiased engineering assessment essential to the prosecution of a credible R&D project.

The estimated total project cost is $787,000. Of this amount, EPA is contributing $500,000. EPA also maintains liaison with project personnel and participates in technical and budget decisions through its assigned Project Officer. Eco, in addition to process design costs, which are not included in the project budget, has contributed $250,000 toward the construction cost of the pilot demonstration plant. The construction cost was $491,000, or roughly 62 percent of the estimated total project cost. The installation history of the Deep Shaft pilot plant will be drawn upon extensively in developing an estimated capital cost for a proposed expanded 37,800-m^3 (10-mgd) facility for Ithaca. Ithaca is providing approximately $37,000 to the R&D project in the form of operating and laboratory personnel time and services.

Visitors' Program

Consulting engineers, municipal personnel, government officials, industry representatives, and the public, both domestic and foreign, are encouraged to visit the Ithaca Deep Shaft demonstration plant. Visitation programs have been scheduled for 2-3 consecutive days during the middle of each month, at which time Eco, City, and/or S&W representatives will conduct guided tours through the facility. Visits to the plant can be confirmed by contacting Eco at telephone number (416)226-7430 or writing to Ithaca Visitors' Program, c/o Eco-Research Ltd., P.O. Box 200, Station A, Willowdale, Ontario, Canada, M2N 5S8 (10).

ENGINEERING AND ECONOMIC CONSIDERATIONS

DESIGN AND OPERATING CRITERIA

General Comments

The Deep Shaft Process departs substantially from conventional activated sludge alternatives in appearance, design criteria, and operational mode. To list the most obvious:

1. It utilizes a vertical rather than a horizontal bioreactor.
2. Its mixed liquor solids tend to float instead of settle.
3. It does not require diffusion equipment or mixers to dissolve oxygen.
4. The microorganisms in its biomass are subjected to wide swings in hydrostatic pressure rather than uniform hydrostatic pressure.
5. It operates as an ultra high-rate suspended growth process with low nominal detention times and highly concentrated biomass.
6. It produces internal oxygen uptake rates up to 8-10 times higher than traditionally accepted values.
7. It reportedly converts some organics directly to carbon dioxide and water, resulting in less excess sludge production than occurs at comparable F/M loadings with surface systems.

8. It produces a 3-8 times more concentrated waste sludge than achieved conventionally.

9. Its minimum air supply requirement to sustain mixing, i.e., shaft circulation at desired velocities, is substantially larger per unit volume than to maintain solids suspension in horizontal bioreactors.

The above deviations from conventional activated sludge practice present difficulties in comparing Deep Shaft with other suspended growth process alternatives using time-honored sanitary engineering analysis techniques. This is particularly true of substrate utilization kinetics, oxygen transfer, power requirements, and excess sludge production. Equitable process design comparison is further hampered in that the preferred feed for Deep Shaft systems in most cases is raw wastewater, while conventional activated sludge systems with the exception of extended aeration are normally mated with primary clarification.

The Ithaca demonstration project as well as several full-scale plants soon to begin operation in Canada are expected to substantially broaden the data base of the Deep Shaft flotation clarification mode. In the interim, the historical and ongoing experiences derived from Eco's pilot facility at Paris, Ontario, continue to serve as the major source of design and operating criteria for the Deep Shaft Process.

Process Comparisons

Suggested design and operating criteria for treating typical municipal wastewater are given in Table 3 for several activated sludge options. The processes compared include the Deep Shaft flotation mode without primary clarification, conventional air activated sludge with and without primary clarification, oxygen activated sludge with and without primary clarification, and extended aeration (no primary clarification).

Typical municipal wastewater was defined for this exercise as having total BOD_5 and suspended solids concentrations of 200 mg/l each. Primary clarification removals of 35 percent for total BOD_5 and 65 percent for suspended solids were assumed. Sufficiently conservative design values were selected to produce an anticipated average secondary effluent quality of 20 mg/l each of total BOD_5 and suspended solids. Extended aeration operated at the conditions shown would also be expected to produce a nitrified effluent.

Table 3 represents an attempt to develop a preliminary process basis for evaluating Deep Shaft against competing alternative technologies. Recommended design values for the air, oxygen, and extended aeration options can be found throughout the literature (13, 14, 15, 16, 17, 18, among others). The ranges shown for these three processes reflect the approximate consensus of the literature, where possible, tempered in some cases with judgment and best estimates. Eco generated reports and correspondence (7, 9, 10, 11, 12, 19, 20) were relied on for the most part in defining parameter ranges for Deep Shaft. In drawing conclusions from Table 3, the reader is cautioned that the design and operating criteria presented for Deep Shaft are much more tentative at this time than for the historical technologies.

TABLE 3. COMPARATIVE DESIGN AND OPERATING CRITERIA OF SELECTED ACTIVATED SLUDGE PROCESSES FOR TREATING TYPICAL MUNICIPAL WASTEWATER

Parameter	Deep Shaft Flotation Mode Without Primary Clarification	Air Activated Sludge		Oxygen Activated Sludge		Extended Aeration (No Primary Clarification)
		Without Primary Clarification	With Primary Clarification	Without Primary Clarification	With Primary Clarification	
Bioreactor						
Nominal detention time, hr	0.5-0.75	6-8	5-7	1.5-2.5	1.25-1.75	18-36
MLSS, mg/l	7000-12,000	2000-3000	1500-2500	4000-6000	3500-5000	3000-6000
% volatile	0.6-0.7	0.65-0.75	0.7-0.8	0.65-0.75	0.7-0.8	0.55-0.65
F/M loading, kg BOD_5/day/kg MLVSS	0.75-1.25	0.3-0.5	0.25-0.45	0.55-0.8	0.5-0.75	0.05-0.15
Volumetric organic loading,						
kg $BOD_5/day/m^3$	5.6-8.0	0.5-0.8	0.4-0.65	2.2-3.2	2.0-2.8	0.16-0.32
lb BOD_5/day/1000 ft^3	350-500	30-50	25-40	135-200	125-175	10-20
Sludge retention time[a], days	2-4	3-6	4-8	1-2	2-3	20-30
Solids Separation Unit						
Surface overflow rate:						
average, $m^3/day/m^2$	20-29	20-29	20-29	18-26	20-29	12-16
gpd/ft^2	500-700	500-700	500-700	450-650	500-700	300-400
peak[b], $m^3/day/m^2$	41-49	41-49	41-49	37-45	41-49	33-41
gpd/ft^2	1000-1200	1000-1200	1000-1200	900-1100	1000-1200	800-1000

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TABLE 3. (continued)

Parameter	Deep Shaft Flotation Mode Without Primary Clarification	Air Activated Sludge		Oxygen Activated Sludge		Extended Aeration (No Primary Clarification)
		Without Primary Clarification	With Primary Clarification	Without Primary Clarification	With Primary Clarification	
Mass loading, kg TSS/day/m^2	293-439	73-122	49-98	146-195	122-171	73-122
lb TSS/day/ft^2	60-90	15-25	10-20	30-40	25-35	15-25
Solids Separation Unit (cont.)						
Return sludge flow rate, % of Q	15-25, float 30-50, bottom	25-45	25-50	30-60	30-70	75-150
Return sludge concentration, % TSS	7-10, float 3-4, bottom	0.8-1.2	0.6-1.0	1.2-2.0	1.0-1.5	0.75-1.0
Air, Oxygen, and Power Requirements						
Air supply rate, m^3/kg BOD_5 removed	6-25^c	50-94^d		-		187-250^f
std. ft^3/lb BOD_5 removed	100-400^c	800-1500^d		-		3000-4000^f
Oxygen utilized, kg/kg BOD_5 removed	2.0-2.4	0.9-1.3		1.0-1.4		1.8-2.2^f
Oxygen transfer efficiency in wastewater, % (O_2 utilized /O_2 supplied)	40-90^c	8-15^d		90-95		8-15^d
Oxygen transfer rate in wastewater,						
kg O_2/wire kWh	0.9-2.7^c	0.9-1.5^d		1.2-1.5^e		0.9-1.5^d
lb O_2/wire hp-hr	1.5-4.5^c	1.5-2.5^d		2.0-2.5^e		1.5-2.5^d
Aeration system power requirement,						
wire kWh/1000 m^3	80-289	112-177	64-112	121-145	72-88	193-321
wire hp-hr/mil gal	500-1800	700-1100	400-700	750-900	450-550	1200-2000

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TABLE 3. (continued)

Parameter	Deep Shaft Flotation Mode Without Primary Clarification	Air Activated Sludge		Oxygen Activated Sludge		Extended Aeration (No Primary Clarification)
		Without Primary Clarification	With Primary Clarification	Without Primary Clarification	With Primary Clarification	
Sludge Production						
Primary sludge TSS^g, g/m^3	-	-	132	-	132	-
lb/mil gal	-	-	1100	-	1100	-
Waste activated sludge:						
VSS, g/m^3	72-90	96-132	54-78	96-120	54-66	36-60
lb/mil gal	600-750	800-1100	450-650	800-1000	450-550	300-500
kg/kg BOD$_5$ removed	0.4-0.5	0.55-0.75	0.5-0.7	0.55-0.65	0.5-0.6	0.2-0.35
TSS, g/m^3	108-138	138-186	72-102	138-168	72-90	60-96
lb/mil gal	900-1150	1150-1550	600-850	1150-1400	600-750	500-800
kg/kg BOD$_5$ removed	0.6-0.75	0.75-1.05	0.65-0.95	0.75-0.95	0.65-0.8	0.35-0.55
Total plant raw + waste sludge TSS, g/m^3	108-138	138-186	204-234	138-168	204-222	60-96
lb/mil gal	900-1150	1150-1550	1700-1950	1150-1400	1700-1850	500-800
Final effluent solids^h:						
VSS, g/m^3	13	14	16	14	16	12
lb/mil gal	110	120	130	120	130	100
TSS, g/m^3	20	20	20	20	20	20
lb/mil gal	170	170	170	170	170	170

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TABLE 3. (continued)

Parameter	Deep Shaft Flotation Mode Without Primary Clarification	Air Activated Sludge		Oxygen Activated Sludge		Extended Aeration (No Primary Clarification)
		Without Primary Clarification	With Primary Clarification	Without Primary Clarification	With Primary Clarification	
Sludge Production (cont.)						
Total sludge production (waste + effluent) in secondary system:						
VSS, g/m^3	85-103	110-146	70-94	110-134	70-82	48-60
lb/mil gal	710-860	920-1220	580-780	920-1120	580-680	400-600
kg/kg BOD$_5$ removed	0.45-0.55	0.6-0.8	0.65-0.85	0.6-0.75	0.65-0.75	0.25-0.4
TSS, g/m^3	128-158	158-206	92-122	158-188	92-110	80-116
lb/mil gal	1070-1320	1320-1720	770-1020	1320-1570	770-920	670-970
kg/kg BOD$_5$ removed	0.7-0.9	0.9-1.15	0.85-1.1	0.9-1.05	0.85-1.0	0.45-0.65

[a] Defined as kg MLSS in bioreactor/(kg TSS lost in waste activated sludge and final effluent/day).

[b] Dry weather peak.

[c] Depends on shaft diameter and degree of air tuning in shaft (Eco II vs. Eco III).

[d] Lower values representative of coarse bubble diffusers; higher values representative of fine bubble diffusers.

[e] Lower values apply to systems employing pressure swing adsorption (PSA) oxygen generation; higher values apply to systems employing cryogenic oxygen generation.

[f] Includes oxygen requirements to sustain nitrification.

[g] Calculated on the basis of 65 percent removal of an assumed raw wastewater concentration of 200 mg/l.

[h] Assumes a final effluent TSS of 20 mg and final effluent solids volatility in accordance with assumed mixed liquor volatility of respective processes.

Projected operating conditions of Deep Shaft that vary markedly from the other activated sludge processes shown include its low bioreactor detention time and inversely high MLSS levels and organic load capacities and the high mass loading capabilities and return sludge concentrations of its solids separation unit. Oxygen activated sludge, which is itself considered a high-rate suspended growth process, requires about three times as much bioreactor volume for equivalent treatment based on Table 3 estimates.

Deep Shaft's uniquely high return sludge solids levels and mass loading characteristics are due, of course, to the first-of-kind substitution of flotation for sedimentation in achieving mainstream liquid/solids clarification. Flotation is the process element that Eco believes is essential to making Deep Shaft economically viable in North America. In addition to approximately doubling the attainable MLSS concentration (and by inference halving required bioreactor volume), flotation operation produces a floated solids concentration sufficiently high as to require no subsequent thickening during waste sludge handling. The potential for eliminating waste sludge thickening altogether will ultimately be decided by the degree of bottom solids wasting required. The thinner bottom solids in large enough quantities could dilute thicker float waste sludge to the point that some form of supplemental thickening would be needed.

The basic parameters governing bioreactor and flotation unit sizing and performance have been fairly well established at Paris and other pilot study locations (7). Further, the results of Ithaca's initial startup period tended to confirm bioreactor and flotation clarifier sizing criteria. The factors relative to Deep Shaft operation that still remain very much in a state of flux involve air/oxygen/power requirements and sludge production. These facets of the Deep Shaft Process are discussed in the following two subsections.

Air and Power Requirements

The data in Table 3 indicate a predicted wide variance for the above Deep Shaft parameters, much more so than for the comparative processes. This wide variance is due largely to shaft diameter and corresponding energy loss attributable to friction. The relationship of organic loading to oxygen transfer based on Eco kinetic models (9) is shown in Figure 10 for the 44-cm (17.25-in.) ID Ithaca pilot shaft. For this particular shaft, projected air requirements for hydraulic circulation are considerably larger than for biological metabolism. The net result of this disparity is relatively low apparent oxygen transfer efficiencies (OTE's) of 38-53 percent in the F/M region (0.75-1.25 kg BOD_5/day/kg MLVSS) of interest for municipal wastewater. Actually, most of the oxygen supplied is dissolved at the bottom of the shaft. However, since biological oxygen demand isn't sufficient to use it all, some of the DO effervesces and is eventually lost from the system.

To illustrate the interdependence of shaft diameter and power requirements, consider that the Ithaca pilot shaft (Eco II design) needs an air supply rate of 22 l/sec (53 scfm), equivalent to 650 kg O_2/day (1434 lb/day), to sustain desired circulation velocities. This is accomplished with a line (or wire) power draw of 11.2 kW (15 hp). Assuming a motor efficiency of 90 percent and an air compressor efficiency of 85 percent, the power delivered to the

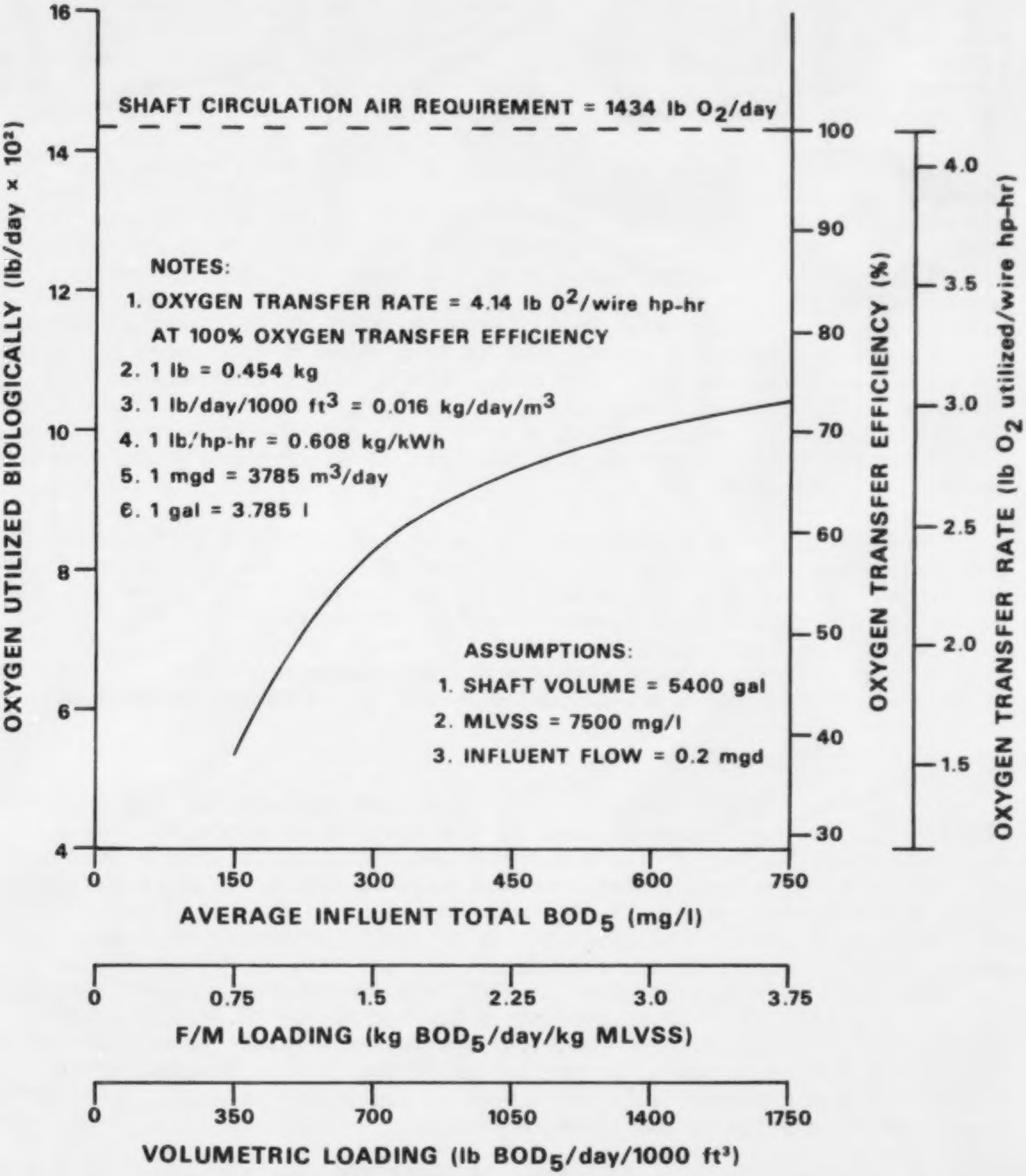

FIGURE 10. OXYGEN TRANSFER - ORGANIC LOADING RELATIONSHIPS FOR ITHACA DEEP SHAFT PILOT SYSTEM

water in the shaft is approximately 8.6 kW (11.5 hp). Given a shaft volume of 20.5 m^3 (724 ft^3), the mixing power requirement for the Ithaca pilot shaft is about 0.42 kW delivered to the water/m^3 (16 hp/1000 ft^3).

By contrast, a 112-kW (150-hp) compressor will be used to supply air for the two 137-cm (54-in.) diameter x 137-m (450-ft) deep shafts at Portage la Prairie up to influent flows of 227,000 m^3/day (6 mgd) (20). Assuming that the line draw proves to be the full 112 kW (150 hp) and using the same motor and compressor efficiencies as Ithaca, the mixing power requirement for this design is only about 0.21 kW delivered to the water/m^3 shaft volume (8 hp/1000 ft^3), or half of the Ithaca pilot shaft requirement. Anticipated apparent OTE at Portage la Prairie is much higher than 38-53 percent. The Portage la Prairie system is being constructed using an Eco III design approach. Eco III, in addition to modifying flotation tank operating procedures, employs a concept Eco calls "air tuning," which is claimed to result in lower net air requirements.

Similarly, a possible Eco I Deep Shaft design proposed in April 1979 (21) for Ithaca's eventual 37,800-m^3 (10-mgd) expansion can be shown to have a projected mixing power requirement of 0.33 kW delivered to the water/m^3 shaft volume (12.5 hp/1000 ft^3). This calculation is based on the proposed use of a 225-kW (302-hp) compressor to supply air to two 152-cm (60-in.) diameter x 187-m (615-ft) deep shafts. Eco now believes that by applying Eco III design concepts, compressor size for a full-scale Ithaca plant could be reduced to 134-149 kW (180-200 hp). If substantiated, proposed mixing power delivered to the water would decrease to around the 0.21-kW/m^3 (8-hp/1000 ft^3) figure projected for Portage la Prairie.

The above discussion emphasizes that minimum mixing power requirements for Deep Shaft can be expected to vary by 100 percent, depending on shaft size and system design. Conversely, the power requirement to adequately mix biomass solids in conventional horizontal tanks has been fairly well established at about 0.013 kW delivered to the water/m^3 tank volume (0.5 hp/1000 ft^3), irrespective of tank geometry. Adjusting tank volume to obtain equivalent treatment with an air activated sludge system, say 6-hr detention time for air activated sludge vs. 40 minutes for Deep Shaft, increases conventional tank mixing requirements to 55-60 percent of that for Deep Shaft.

Another factor that must be considered in defining power requirements is biological oxygen demand. Whereas air requirements for mixing would be sufficient to satisfy Deep Shaft oxygen demand with most municipal wastewaters, this is not the case in many conventional activated sludge designs. For example, coarse bubble air diffuser systems can consume 0.039 kW delivered to the water/m^3 (1.5 hp/1000 ft^3) or more to satisfy oxygen demand. Fine bubble air diffusers and oxygen activated sludge transfer equipment more nearly match biological power with mixing power requirements.

The preceding discussion suggests that careful engineering analysis will be essential to equitable comparison of aeration power requirements for Deep Shaft with conventional technology. On the basis of the limited and somewhat speculative information now available, Table 3 indicates that Deep Shaft aeration system power requirements can range anywhere from decided economic attractiveness to being just barely competitive with extended aeration.

Oxygen Utilization and Sludge Production

A claimed Deep Shaft operating characteristic that defies accepted biological process behavior concerns excess sludge production. Total biological sludge production for the Deep Shaft Process is estimated in Table 3 at 0.45-0.55 kg VSS/kg total BOD_5 removed. Corresponding estimates for conventional air and oxygen activated sludge processes are 0.6-0.85 and 0.6-0.75 kg VSS/kg total BOD_5 removed, respectively. The issue is not that Deep Shaft estimated sludge production is less than that of conventional suspended growth options, but that such claims should be made in view of the exceedingly high F/M loadings that are imposed on the process.

Traditionally, biological kinetic theory has held that in the absence of oxygen deficiency a direct relationship exists between F/M loading and sludge production. Such a relationship explains, for instance, the low sludge yield of extended aeration units. The equal to slightly less predicted yield of oxygen activated sludge vs. air activated sludge in Table 3 despite the former's much higher (approximately double) typical F/M loading has been variously attributed to improved oxygen penetration of microfloc (22) and/or the large amount of sludge inventory routinely maintained in some oxygen final clarifiers (23). With the latter hypothesis, oxygen activated sludge F/M loadings based only on aerator sludge inventory are represented as artifacts. The smaller actual F/M loadings, calculated taking total system sludge inventory into account, reportedly fall into line with prevailing sludge production theory.

Reported Deep Shaft sludge yield does not agree with the above traditional relationship. The anticipated yield at an F/M loading of 1.26 kg BOD_5/day/kg MLVSS with conventional processes would be expected to approach 1 kg VSS/kg BOD_5 removed and to exceed 1 kg TSS/kg BOD_5 removed, rather than the 0.5 kg/kg BOD_5 removed yield (7) suggested on the basis of the Billingham study.

Eco's explanation for the observed low sludge yield both at Billingham and Paris is that it ....."hypothetically results from extremely high oxygen uptake rates (200-250 mg O_2/g VSS/hr) occurring in the vicinity of the D_2R_1 influent wastewater injection point, which stimulates direct conversion of organic carbon to carbon dioxide and water and bypasses the customary cell synthesis process" (20). Figure 10 suggests that something unusual is happening in the shaft in terms of unit oxygen utilization (consumption) that may relate to sludge yield. The Eco kinetic models on which Figure 10 is based predict that in the F/M range of 0.75-1.25 kg BOD_5/day/kg MLVSS, oxygen utilization varies from 2.4-2.0 kg/kg BOD_5 removed, assuming a total BOD_5 removal of 90 percent. This is roughly twice the utilization rate normally encountered in biological systems that are not nitrifying.

High oxygen utilization rates would seem to be tied to the claimed high oxygen uptake or respiration rates. If in fact these rates of oxygen utilization can be substantiated, the plausibility of reported low sludge yields would be strengthened, either through auto-oxidation of cell mass or through some other unexplained metabolic pathway.

COST CONSIDERATIONS

Factors Affecting Costs

The Deep Shaft Process is characterized by a number of potential cost savings attributes, site specific cost features, and currently indeterminate economic considerations that taken together preclude comprehensive cost analysis for generalized comparisons at this time.

Biological reactor detention times in the order of 40 minutes are one-ninth to one-thirty sixth of the typical required reactor volumes of conventional air and extended aeration plants. Shaft placement costs can, however, be thirty times more expensive per unit volume than a surface system (7).

Land area required for Deep Shaft plants can be as little as 25 to 35 percent of land area required for conventional plants. In addition to land cost savings, this characteristic of reduced size suggests applicability for plant expansion plans in areas of limited land availability or wherever housing the plant facilities is an important consideration.

For municipal applications, aeration requirements will generally be established by hydraulic circulation requirements rather than organic load considerations. The power requirement to achieve minimum acceptable hydraulic circulation rates of 1-1.5 m/sec (3-5 ft/sec) is a function of the friction loss in the shaft, which is a function of shaft diameter.

Shaft diameter, ranging from 0.5 to 10 m (1.5 to 33 ft), together with a depth determination, ranging from 100 to 200 (325 to 650 ft), establishes the reactor volume and detention time for a given flow and circulation rate. Shaft diameter also determines the shaft placement technology. Sandford et al. (7) provided useful insight on shaft placement technology. Shafts with a diameter of less than 3.7 m (12 ft) are usually drilled. Shafts larger than 5.5 m (18 ft) are usually mined. Shaft placement technology is well established using conventional oil and water well drilling technology with over 2,500,000 oil wells being placed in the past 25 years. All shafts are encased for their full length with either steel or concrete and grouted to the subsurface geology using sulfate resistant cement. Sandford further reports, "Shaft placement technology is also sufficiently developed to allow, with confidence, the installation of shafts in earthquake and slide areas."

The use of flotation clarification further contributes to reduced land requirements through potential elimination of post-clarification thickening, reduced recycle volumetric requirements, and higher mass loading capabilities for the clarification process.

Cost factors which are indeterminate at this time include: (1) the amount of polymer, if any, that is necessary to achieve effluent quality objectives and stable flotation performance and (2) the amount of waste sludge produced per unit of BOD_5 removed.

Dunlop (24) provides a useful overview of the characteristics of the sludge produced by the Deep Shaft Process. A summary of his findings are shown in Table 4.

TABLE 4. CHARACTERISTICS OF DEEP SHAFT SLUDGE

Parameter	Deep Shaft Sludge	Conventional Activated Sludge
Adenosine triphosphate content, μg/ml	0.8062	0.9908 Billingham 0.5368 Durham
Sludge specific activity, mg O_2/g MLSS/hr	32 - 46	15 - 58
K_s determination (Michaelis-Menten plot), mg/l	50	20 - 50
Floc size, microns	30 - 100	30 - 1800
Protozoal population	No change	
Capillary suction time, sec	50 - 100	200 - 400 (digested primary)
Specific resistance to filtration, 10^{14} in./kg	1.44	8.54 (0.5 to 2 normal)
Compressibility	0.85	0.78

These results indicate that there is no major difference in the characteristics of Deep Shaft sludge compared to conventional sludges. The six-fold decrease in specific resistance measured experimentally was statistically significant, but the Deep Shaft value is within the range of reported values for conventional sludges. The capillary suction time of 50-100 sec suggests a good dewatering sludge.

It is interesting to note that the specific activity reported by Dunlop for Deep Shaft sludge is 4-5 times lower than the 200-250 mg O_2/g VSS/hr oxygen uptake rates claimed by Eco. The discrepancy is believed to be due to differing environmental conditions. Eco's values apply to sludge exposed to the high oxygen transfer intensity existing within an operating shaft. Dunlop's data were developed by laboratory methodology on sludge withdrawn from a shaft. Eco's position is that ordinary microbes respond favorably to high aeration intensity and exhibit oxygen uptake rates in the shaft that cannot be duplicated in standard laboratory tests (20).

Cost Estimates

Thompson (19) published cost estimates for Deep Shaft and extended aeration for the City of Portage la Prairie, Manitoba, in March 1979. These costs are compared in Table 5 for capital and in Table 6 for operation and maintenance (O&M). The plant is designed to treat a dry weather flow of 14,300 m^3/day (3.75 mgd)(peak wet weather flow of 37,800 m^3/day = 10 mgd) with a combined domestic/industrial organic load of 5440 kg BOD_5/day (12,000 lb/day).

TABLE 5. CITY OF PORTAGE LA PRAIRIE CAPITAL COST COMPARISON

Unit Process	Deep Shaft	Extended Aeration Using Jet Aerators
Aeration	-	$430,000
Final clarification	$103,000	830,600
Head works (screening, degritting)	904,600	980,600
Outside services (roads, piping)	165,000	276,100
Eco (fixed price including shaft, compressors, piping, flotation tank, mechanical equipment, and instrumentation)	1,750,000	
Engineering	201,000	272,000
Total	$3,123,600	$2,789,300

The consultant recommended Deep Shaft on the basis of lower operating costs, the offer by Eco-Research Ltd. of a firm price for construction costs, the significantly lower electrical requirements (a rapidly increasing cost component), the ability to easily house the facility, and the improved disposal characteristics of the flotation sludge.

In September 1977, Sandford (7) reported the capital costs of Deep Shaft facilities treating 2270 m^3/day (0.6 mgd) and 90,800 m^3/day (24 mgd) of domestic wastes and 13,600 m^3/day (3.6 mgd) of combined domestic/industrial wastes as being \$0.5 million, \$6.5 million, and \$2.0 million, respectively. These cost estimates included installation of the shaft and solids separation system, all compression equipment, process control equipment, piping, and civil, electrical, mechanical, and painting work required. They did not include preliminary treatment (screening, degritting, headworks, etc.), post-treatment (disinfection, sludge dewatering), or any buildings.

TABLE 6. CITY OF PORTAGE LA PRAIRIE OPERATION AND MAINTENANCE COST COMPARISON

Unit Process	Deep Shaft	Extended Aeration Using Jet Aerators
Electricity	$28,486	$56,971
Chemicals	62,850	30,000
Sludge hauling	48,667	60,833
Salaries	40,000	40,000
Maintenance	52,000	52,000
Fuel	6,500	6,500
Subtotal O&M	238,503	246,304
Amortization, 20 yr @10%	366,899	330,215
TOTAL ANNUAL COST	$605,402	$576,519

Preliminary cost estimates were prepared in a 1979 facilities planning exercise comparing 13 alternative treatment systems for a proposed 37,800-m^3/day (10-mgd) plant expansion at Ithaca, New York (25). The estimates indicated that Deep Shaft was competitive, both on the basis of capital and O&M costs, with the two or three most cost-effective conventional alternatives. The estimators acknowledged that several important cost factors could not be confidently assessed for the Deep Shaft Process at that time. Cost sensitive features of Deep Shaft that will ultimately determine its full-scale economic attractiveness for Ithaca include: (1) the degree to which the process must or should be housed and the required quality of that housing and associated utility services, (2) the quantity of flotation aid chemicals required, if any, (3) excess sludge production, (4) full-scale power requirements, (5) the feasibility of eliminating sludge thickening equipment, (6) geologic conditions and associated drilling costs, (7) the degree of permissable and/or realistically achievable design flexibility to reduce shaft and flotation unit redundancy, and (8) the level of operating sophistication and attention required.

CURRENT STATUS OF DEEP SHAFT IMPLEMENTATION

NORTH AMERICAN IMPLEMENTATION

Eco's market activities in North America to date have been primarily relegated to Canada. As indicated in Table 7, three full-scale Deep Shaft plants are

TABLE 7. DEEP SHAFT PLANTS IN NORTH AMERICA

Location	Type of Plant	Type of Wastewater	Influent BOD_5, mg/l	Design Flow m^3/day	Design Flow mgd	Status as of March 1980
Virden, Manitoba, Canada	Full scale, Eco I design	Municipal	200	2400	0.63	Operating (startup)
Portage la Prairie, Manitoba, Canada	Full scale, Eco III design	Municipal/ food processing wastes	400	14,200	3.75	Under construction
Molson Breweries Ltd., Barrie, Ontario, Canada	Full scale, Eco II design	Brewery wastes	2400	2180	0.58	Operating (startup)
Ithaca, New York, U.S.A.	Demonstration, Eco II design	Municipal	100-175	757	0.20	Operating (startup)
Paris, Ontario, Canada	Experimental, Eco III as of April 1980	Municipal/ textile wastes	150-200	454	1.20	Operating (study completed)
Molson Breweries Ltd., Barrie, Ontario, Canada	Pilot, Eco II design	Brewery wastes	1000-6000	*	*	Study completed
Anheuser-Busch Inc., Williamsburg, Virginia, U.S.A.	U-tube pilot, Eco I design	Brewery wastes	1500-2000	*	*	Study completed
Agropur Dairy Ltd., Notre-Dame-de-bon-Conseil, Quebec, Canada	U-tube pilot, Eco I design	Dairy wastes	1000	*	*	Study completed

* Denotes the use of variable loadings to determine organic stress-performance relationships.

presently either being started up or constructed in Canada (9). The other five locations listed in Table 7 encompass various size development studies ranging from small U-tube pilot plants to the 757 m^3/day (0.2-mgd) demonstration facility at Ithaca (9).

Deep Shaft technology is applicable to a variety of wastewater treatment situations. Because of its high oxygen intensity and transfer characteristics as well as its fixed air supply requirements for fluid circulation, Deep Shaft will probably find its greatest cost effectiveness in treating high strength industrial wastes. Eco's initial industrial contacts have been with two breweries and one dairy. The full-scale Molson brewery waste treatment facility is an outgrowth of an earlier pilot study at the same site.

The cost attractiveness of Deep Shaft for municipal treatment should be enhanced as the fraction of industrial contribution increases, provided the type of wastes being discharged into the municipal sewer system do not interfere with flotation efficiency. Portage la Prairie (plant scheduled for startup in November 1980) is an example of a city with a relatively strong combined municipal/industrial wastewater that should be well suited to Deep Shaft treatment.

IMPLEMENTATION OUTSIDE NORTH AMERICA

Implementation of the Deep Shaft Process outside North America has involved the United Kingdom, Europe, and Japan, as summarized in Table 8 (9). The U.K. and European plants represent the realization of Deep Shaft marketing efforts by the parent company, ICI, Billingham, England. The two pilot studies in Japan, one completed and one still in progress, were initiated through the offices of ICI Japan.

In contrast to the North American Deep Shaft pilot and full-scale installations, all of which are of Eco design and utilize flotation clarification, the U.K., European, and Japanese facilities employ the ICI flowsheet, i.e., gravity clarification preceeded by vacuum degassing. The emphasis on industrial clients is noticeably higher in the U.K. and Europe than in North America. Eco's decision to substitute flotation clarification for gravity clarification was based partly on: (1) their perceived larger potential municipal market in North America and (2) the accompanying need to reduce shaft volume and associated costs to enhance Deep Shaft's competitive position in treating the high volume-low strength municipal wastewaters typically found in Canada and the United States.

A related technology that should be briefly mentioned is the U-Tube Aeration System marketed in Japan by Kubota Ltd. While similar to the Deep Shaft Process in that it utilizes a vertical, drilled-shaft bioreactor, Kubota's system employs liquid pumping rather than compressed air injection to provide the motive force for circulating mixed liquor.

TABLE 8. DEEP SHAFT PLANTS OUTSIDE NORTH AMERICA

Location	Type of Plant	Type Of Wastewater	Influent BOD_5, mg/l	Design Flow m^3/day	Design Flow mgd	Status as of March 1980
Emslandstarke Co., Emlichheim, West Germany	Full scale, ICI design	Potato processing wastes	2000	1050	0.28	Operating
Anglian Water Authority, Marsh Farm, Thurrock, U.K	Full scale, ICI design	Cornstarch (50%+), municipal (20%), misc. industrial (30%)	1060	6650 (max.)	1.76	Operating
Kimberly Clark Co., Prudhoe, U.K	Full scale, ICI design	De-inking mill wastes	1000	21,600	5.71	Operating
ICI Agricultural Division, Pruteen, U.K.	Full scale, ICI design	Single cell protein production wastes	-	1575	0.42	Periodic operation
Leer, West Germany	Full scale, ICI design	Mixed municipal/ dairy/industrial	400	32,500	8.58	Under construction
Billingham, U.K.	Experimental, ICI design	Municipal	200	363	0.096	Study completed
Holdenhurst, U.K.	Pilot, ICI design	Municipal	-	*	*	Operating
Undisclosed, Japan	Pilot, ICI design	Chemical wastes (organic chlorides)	3000	*	*	Operating
Toyonaka, Japan	Pilot, ICI design	Municipal	200-600	2400	0.63	Study completed
ICI Petrochemical Division, Wilton, U.K.	Pilot, ICI design	Chemical wastes (terephthalic acid manufacturing)	4500	*	*	Study completed; full-scale plant planned
ICI Petrochemical Division, Wilton, U.K.	Pilot, ICI design	Chemical wastes (phenol and phenolics)	2400	*	*	Operating

* Denotes the use of variable loadings to determine organic stress-performance relationships.

SUMMARY

This paper has presented: (1) a review of the United Kingdom origins of the Deep Shaft Process, (2) a synopsis of the development program undertaken in Canada by Eco-Research Ltd. to refine the technology for marketing in North America, (3) a description of EPA's R&D efforts currently underway to ascertain the applicability and cost effectiveness of the process for use in the United States, and (4) an overview of the critical design and operating features requiring further evaluation that will ultimately determine Deep Shaft's role in the wastewater treatment field.

The development of the Deep Shaft Process has been rapid. Eight full-scale treatment plants are either in operation or under construction in Canada, West Germany, and the United Kingdom. At least five different process designs or configurations (ICI, Eco I, Eco II, Eco III, and Kubota) have evolved in the brief 6-year period since the first pilot facility was started at Billingham, England, in 1974.

Penetration of the U.S. municipal market will depend in large measure on the outcome of the Ithaca, New York demonstration project, as well as the economic and risk assumption incentives of the innovative and alternative funding provisions of the Clean Water Act Amendments of 1977 (1). At this point in time, Deep Shaft is considered to represent a novel and potentially innovative alternative to conventional suspended growth biological treatment. Additional operating and performance data must be generated before comprehensive engineering comparisons and an assessment of the process' innovative status can be made. The Ithaca project is expected to contribute significantly to a fuller understanding and characterization of both the flotation and gravity clarification modes of liquid/solids separation and their effect on cost and energy utilization estimates.

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CURRENT DESIGN AND OPERATING EXPERIENCE WITH ANAEROBIC SLUDGE DIGESTION

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I. INTRODUCTION

For the past five decades, the Sanitation Districts of Los Angeles County have successfully designed and operated anaerobic sludge digestion systems. The Districts' experience until recently has been primarily limited to the digestion of primary sludge. In July 1975, the California Regional Water Quality Control Board, in accordance with revised federal waste discharge requirements, mandated the staged installation of secondary treatment at the Joint Water Pollution Control Plant (JWPCP) in Carson, California. The Districts are currently constructing 8.8 m^3/s (200 mgd) of high purity oxygen activated sludge treatment facilities at JWPCP. Based on extensive field pilot plant research studies(1), a sludge processing scheme that includes anaerobic sludge digestion was chosen by the Districts to treat the waste activated sludge (WAS) from the oxygen activated sludge units. This paper discusses the following:

- The basic advantages of anaerobic sludge digestion to the Districts.
- The design parameters and important construction features of the digesters presently under construction.
- The current operating principles and process control procedures for the digestion systems at JWPCP.

A. EXISTING FACILITIES

The Joint Water Pollution Control Plant currently provides advanced primary treatment for 15 m^3/s (350 mgd) of domestic and industrial wastewater. A site layout for the major treatment works discussed in this paper is shown in Figure 1. The plant treats approximately 75 percent of the flow in the entire Sanitation Districts' system, which serves a population of four million. The effluent from the plant is discharged to the Pacific Ocean through deep ocean outfalls 3.2 km (two miles) off the coast. Five other treatment plants in the system return their sludge to sewers which terminate at JWPCP, so the plant serves as a central solids processing facility for the whole system.

The following is a brief summary of the flow diagram for JWPCP as shown in Figure 2. After passing through the inlet works and grit chambers, the wastewater flows through fifty-two primary settling tanks. The settled sludge and concentrated floatables from the primary tanks are pumped to digesters for anaerobic biological treatment. JWPCP has two digestion systems, one comprised of rectangular digesters and one of circular, which are shown in Figure 1. The digesters have a combined capacity of 249,000 m^3 (8.78

million cu ft) and presently treat a primary sludge flow that averages 9500 m^3 (2.5 million gallons) each day, as shown in Figure 17. After digestion the sludge is dewatered by centrifuges and the wet cake is either air-dried and composted on open fields or truck hauled to a landfill for disposal. The composted solids are sold to a local fertilizer company for use as a soil conditioner and fertilizer base.

B. SECONDARY TREATMENT FACILITIES

Construction is in progress for the secondary treatment facilities, Stages I and II, which are shown in Figure 1. Presently the contracts awarded for both secondary treatment facilities and waste activated sludge processing facilities total almost 100 million dollars. Of this total, over 21.5 million dollars in construction contracts have been awarded recently for facilities for flotation thickening and anaerobic digestion of the waste activated sludge. Low speed scroll centrifuges have been selected for dewatering the digested waste activated sludge and design of the facility has begun.

II. ADVANTAGES OF ANAEROBIC DIGESTION

The basic advantages of anaerobic sludge digeston to the Districts are listed and briefly discussed thereafter.

- Energy Recovery
- Sludge Storage and Equalization
- Reduced Odor Potential
- Reduced Health Risk

The anaerobic digestion systems at JWPCP presently produce 160,000 m^3 (5.65 million cu ft) per day of digester gas of which 50 percent is currently utilized in the plant. Digester gas is used by reciprocating engines to pump primary effluent to the ocean and in the future to secondary treatment, and to generate approximately 50 percent of the electrical power used at JWPCP. Digester gas is also used by boilers for digester heating. Presently the remaining 50 percent of the gas is sold to a refinery adjacent to JWPCP.

The existing power plant is being replaced by a combined cycle total energy plant (under design) which will increase the utilization of the digester gas from the present 50 percent to 100 percent, as shown in Figure 3(2). Of equal importance, the overall thermal efficiency obtained from the gas will increase from approximately 23 percent at the present time to greater than 60 percent. This increase in thermal efficiency is achieved by using more efficient prime movers and by recovering the waste heat for digester heating, and miscellaneous building heating and cooling. This efficient and complete utilization of the digester gas will provide almost 80 percent of the total energy requirements for the entire JWPCP after completion of the present 8.8 m^3/s (200 mgd) secondary treatment expansion.

Since a minimum hydraulic detention time must be provided for the anaerobic biological process, sludge is stored in the digestion tanks for a considerable number of days. This storage provides equalization of both sludge characteristics and sludge flow. The benefits of equalization on downstream sludge processes, such as dewatering, are significant in terms of economic savings, reduced operational control requirements and increased performance.

Additional sludge storage capacity in excess of design is available in the digesters at JWPCP by temporary use of gas space. Figure 13 shows that a 1.2 m (4 ft) gas space exists between the sludge level in the tank and the roof. Up to 0.6 m (2 ft) of this gas space can be used for temporary sludge storage during unscheduled downtime of the sludge dewatering facility and will provide up to 16,000 m^3 (565,000 cu ft) of additional capacity. Without this large reserve capacity, excessive storage of sludge in the primary settling tanks would become necessary with resulting deterioration in primary effluent quality and operational problems with sludge pumping from the primary tanks.

The reduced odor potential from digested sludge has allowed the Sanitation Districts to air-dry and compost approximately one-third of its dewatered sludge on open fields at JWPCP. The operation is presently limited to the site shown in Figure 2 because of space available and a need for further improvements in odor reduction and odor control during the composting operation.

The temporary storage of dewatered digested sludge is needed to eliminate the need for around the clock hauling to the landfill and to provide wet weather storage for the composting operations. In conjunction with a minimal chemical scrubbing system, odor release has not been a problem from digested sludge stored in large silos at the sludge dewatering facility.

Compared with the handling of raw primary sludge, digested sludge provides a reduced health risk to the public and especially to employees since the anaerobic digestion process reduces pathogen levels(3).

III. DESIGN PARAMETERS AND CONSTRUCTION FEATURES

A. MASTER PLAN FOR CIRCULAR DIGESTERS

In the late 1960s, an expansion of the anaerobic digestion system at JWPCP was planned to handle the increased sludge flow from additional primary sedimentation tanks and from planned future plant expansions.

The circular digestion system was chosen to be identical in process, operation and control to the rectangular digestion system which operates at high organic loading rates. A simplified process flow schematic of the circular digestion system is shown in Figure 4 and consists of digestion tanks completely mixed by gas recirculation, uniformly heated by direct steam injection, and uniformly loaded by an automated sludge feed system. The sizing and design criteria for the digestion tanks and individual systems are summarized in Table 1 and discussed below.

The resulting master plan for the circular digestion system is shown in Figure 5 and consists of circular digestion tanks and sludge pump stations interconnected by pipe galleries, and a centralized control building which also houses the boilers used for digester heating.

The first six of the circular digesters are in operation and presently digest primary sludge. The remaining six digesters, which complete the master plan, are under construction and will provide an additional 85,000 m^3 (3 million cu ft) of digester capacity to treat an average thickened waste activated sludge flow from secondary treatment of approximately 0.044 m^3/s (1 mgd).

B. DIGESTER SIZING

The design geometry, size, and number of digestion tanks were selected after an evaluation of the process requirements, economics, and site limitations. The design selected provided the maximum total digester volume within a limited area at minimum cost.

An average hydraulic detention time of 15 days was selected for the first six circular digesters. This detention time is considered conservative design for primary sludge digestion at JWPCP, since successful operation at detention times as low as 10 days has been achieved at high organic loading rates. Successful operation is defined as the operation of the digesters with a consistent volatile solids destruction of 50 percent or greater. The volatile solids loading rate chosen per unit volume of design capacity was 2.4 $kg/m^3/day$ (0.15 lb/cu ft/day), based on a 5 percent raw primary sludge solids concentration with 70 percent volatility. This was again conservative design because rectangular digesters at JWPCP have been successfully operated at loading rates up to 6.4 $kg/m^3/day$ (0.40 lb/cu ft/day). For waste activated sludge digestion, the design detention time and solids loading rate are listed in Table 1.

Deep tanks were selected to minimize scum formation. By minimizing the tank surface area through which the digester gas is released, the maximum surface turbulence is developed while scum formation is minimized.

A circular design geometry was chosen with each digester having a volume of approximately 14,160 m^3 (500,000 cu ft), an inside diameter of 38 m (125 ft), a side water depth of 11 m (36 ft) and a maximum water depth of 15 m (50 ft). The circular tanks selected by the Districts' design staff were the largest diameter and the deepest tanks which had previously been constructed as digesters in the United States. Twelve of the tanks conveniently fit the site available and were selected for the master plan as shown in Figure 4.

C. MIXING SYSTEM

Continuous and complete mixing is of prime importance in successful anaerobic digester operation at high organic loading rates. It performs several distinct functions, including:

- Providing intimate sludge-organism contact.
- Insuring uniform temperature distribution.
- Assuring by-product and buffering distribution.
- Minimizing scum formation.
- Minimizing bottom deposits.
- Preventing stratification.

Mixing can be accomplished in digesters either mechanically, hydraulically or by gas recirculation. Mechanical mixers are usually propeller-type of either upflow or downflow design and may be reversible. Hydraulic mixing is accomplished by external pumps which take suction from various lower elevations or locations and discharge at various points at or above the liquid surface. Gas mixing is used by the Districts and has become the most commonly used type of mixing in the United States. Various systems are used to compress the digester gas and inject it at some depth below the liquid surface.

Mixing may be continuous or intermittent, with continuous mixing practiced by the Sanitation Districts. Intermittent mixing is acceptable only when digesters are heavily loaded and the resulting gas production is sufficient to keep the tank adequately mixed. Even in this case, other factors such as erratic loading can trigger a partial upset which might have been avoided with continuous mixing.

The circular digesters at JWPCP are mixed by gas recirculation through internal draft tubes, as shown in Figure 13. Each gas mixer in the circular digesters consists of a 1.2 m (4 ft) diameter steel draft tube with gas diffusion heads at a submergence of 3.7 m (12 ft), and a surface baffle plate to provide for energy recovery. The gas mixers for the circular tanks are similar in design but larger than the gas mixers used in the rectangular digesters at JWPCP.

Prior to design of the circular digesters, a visual model study was conducted at the California Institute of Technology to investigate different multicellular configurations of gas mixer units. Because it was not possible to accurately model gas recirculation rates and establish power requirements in this laboratory investigation, two different mixing unit configurations were incorporated into the digester design for full scale evaluation. Tank No. 1 was constructed with 9 gas mixer units while Tank Nos. 2, 3, and 4 were each constructed with 5 gas mixer units with provisions for four additional mixer units. Three gas compressors were installed on Tank Nos. 1-4 to allow for varying the total gas recirculation rate between 1.1 and 3.8 m^3/s (2400 to 8100 cfm).

During 1973 and 1974 the Districts conducted a full scale field investigation on Tank Nos. 1-4 to compare the two mixing designs and to determine the effect on mixing efficiency of gas recirculation rate and number of gas mixer units. The study included the following:

- Measurement of active digester volume and the time required to achieve complete mixing of the digester contents using a fluoride salt tracer.

- Measurement of the depth of bottom deposits in the digesters with respect to time.

- Monitoring of gas production and close control of digester loading rates.

Based on the results of this study, it was concluded that 1) the future circular digesters should each be designed with a total of 5 gas mixer units and 2) all existing and future circular digesters should be operated at a gas recirculation rate of 1.4 m^3/s (2900 cfm)(4). Figures 6, 7, and 8 show the recommended mixing system design for the circular digesters.

Probably the most significant observation of this study was that gas recirculation rates in excess of 3.4 m^3/s (7300 cfm) did not prevent the buildup of bottom deposits. The study also determined that the mixing system design recommended for the circular digesters provides an individual cell turnover time of about 24 minutes and a complete tank turnover time of 2 to 3 hours.

D. HEATING SYSTEM

The purpose of the heating system is to raise the temperature of the sludge to a range which provides the optimum environment for the organisms. Digesters are commonly operated in either the mesophilic range (30°C to 38°C) or the thermophilic range (45°C to 57°C). Either range produces satisfactory digestion, however difficulties in maintaining the higher temperatures and some reported problems of poor solids-liquid separation have resulted in the more common use of the mesophilic range. Provision of heat to maintain the proper temperature may be accomplished by external or internal heat exchangers or by direct steam injection.

The digesters at JWPCP are presently heated by direct steam injection which is injected in the draft tube of the mixer units, as shown in Figure 6. Low pressure steam from the boilers is piped to each mixer unit where it is measured and then discharged through a separate submerged steam injection line in the middle of the draft tube. Because the gas mixing system is efficient, uniform heating is normally achieved by adding steam through only one or two gas mixer units, usually the units closest to the sludge feed points.

In the future, waste heat from the previously mentioned combined cycle total energy plant will be used to heat all the digesters at JWPCP. Digesters will either continue to be heated by direct steam injection, using steam produced from waste heat boilers; or by external spiral-type sludge heat exchangers, using low level condenser waste heat from a steam turbine.

E. FEED SYSTEM

Continuous or frequent incremental feeding is vital to satisfactory operation. Feeding is accomplished at JWPCP by adding sludge to each digester several times each day. This is not so important from the standpoint of possible intermittent over or under feeding, as it is to equalize variations in sludge concentration, sludge characteristics, and industrial solids. If each digester is fed only once each day, the sludge fed to any one tank may differ greatly from that fed to another tank. Thus by incremental feeding, these variations are equalized.

Maximum solids concentration is essential for good digester operation. Providing as thick a feed sludge as possible provides for maximum detention time, lowers heating requirements, and prevents dilution of buffering capacity and the excessive loss of seed organisms. The concentration of solids in the sludge feed directly affects the digester detention time and loading relationship. For a given digester volume, the only way to increase either detention time or loading rate without decreasing the other, is to increase the solids concentration in the feed sludge.

Sludge thickening may be accomplished either in the settling tanks or in separate thickening facilities. Sludge blanket control, density meters and timer controlled positive displacement pumps have been used successfully to thicken the sludge in settling tanks. Separate sludge thickening facilities typically use flotation processes, centrifuges, or vacuum filters. If both primary and secondary sludges are involved, a combination of these thickening devices may be necessary.

A uniform sludge solids concentration is needed at JWPCP throughout the day to allow the digesters to be fed on a volumetric basis, thus permitting easy measurement and control. For the raw primary sludge this is achieved by intermittent sludge pumping, which is automatically controlled by nuclear density gauges in conjunction with timers. A single large capacity centrifugal pump withdraws sludge in sequence from four to six primary tanks, as shown in Figure 9. Timer controls initiate the pumping process, while a density controller terminates the pumping process and provides a continuous monitoring of the solids concentration.

The waste activated sludge (WAS) from the secondary treatment facilites at JWPCP will be thickened to a minimum 3.5 percent by dissolved air flotation prior to digester feeding, as shown in Figure 2. Polymer addition to the flotation process will insure that a uniform and concentrated waste activated sludge is fed to the digesters.

The uniform control of raw primary sludge concentration at JWPCP has allowed digester loading to be successfully controlled by volumetric means using automatic incremental feed control systems. The two independent sludge feed control systems for the circular digesters are shown schematically in Figure 10. Each digester can receive primary sludge or thickened waste activated sludge (TWAS) in combination or separately, and is normally fed each sludge in 10 equal increments each day. The volume of sludge fed to each digester per feeding is manually set on individual incremental feed counters. Different digester loadings are achieved by different settings of these incremental feed counters.

The following is a brief description of the sequential feeding operation for the circular digesters, as shown in Figure 10. Both raw primary sludge and TWAS are pumped through separate piping systems and discharged into the top of each digester, as shown in Figure 11. The digester feed pumps start and stop automatically, based on sludge levels in their respective wetwells, and pump at constant flow rates. When a digester has been fed the volume of sludge set on its incremental feed counter, the feed valve to the next digester in the sequence is automatically opened and the preceding feed valve is automatically closed. In the same manner as described above, the individual control system for both sludges will feed each digester the amount indicated on its incremental feed counter. After completion of each cycle through the digesters, the control system switches back to the beginning of its sequence and repeats the cycle beginning with the first digester. The feed control systems for both the rectangular and circular digesters feed all the sludge in preset proportions to all the digesters in service.

With good uniform control of solids concentration, incremental feeding, volumetric measurement, and good sampling to determine average solids concentration; the sludge loading of individual digesters at JWPCP is controlled at desired rates from day to day.

F. STRUCTURAL DESIGN FEATURES

The major structural design features for the circular digesters at JWPCP are described below.

1. Roofs. Digester roofs are the fixed type with flat slab reinforced concrete construction. Each roof is supported by an exterior wall at its perimeter and by interior reinforced concrete columns, as shown in Figures 12 and 13. The roof is designed to support gas compressor equipment and piping, and sits on a continuous neoprene pad at the wall to allow for movement of the wall.

Since the roof is gas tight, it is protected against structural damage by gas seals which are designed to vent the digester gas in the tank to the atmosphere if the gas system pressure rises too high. Each digester has four gas seals which are essentially U-tubes filled with water and are located as shown in Figure 16.

2. Walls. The circular tank walls, shown in Figure 13, are prestressed concrete, vertically post-tensioned, and horizontally wrapped on the outside

face with prestressing strands covered with gunite. The walls are supported on neoprene bearing pads to allow for movement during post-tensioning and filling of the tank.

The digestion tank design drawings for all twelve digesters also incorporated a reinforced concrete wall design as an alternate to the prestressed wall design. Although never constructed, this alternate design insured competitive bidding for the tank wall construction, a major cost item, and resulted in economical construction of the digestion tanks.

3. Bottoms. Tank bottoms in the circular digesters are constructed of reinforced concrete and are of two different designs. Tank Nos. 1-6 have conical shaped bottoms, which the Districts' operation staff has found difficult to clean. As a result, the bottom design was changed to a hopper configuration for the six tanks under construction. The hopper bottom design is shown in Figure 12 and its design features are discussed below.

G. CLEANOUT SYSTEM

Over a period of time, scum and grit accumulate in digesters and reduce the volume available for active digestion. Without removal of these items, the active volume is eventually reduced to the point where overloading occurs. Scum consists of a floating mat of sludge, grease, rags, plastic, and other materials. Slowing the rate of accumulation is achieved primarily by adequate mixing.

A portion of the grit which passes through the plant's grit removal facilities accumulates in the digesters, constantly decreasing the active volume. This in turn places an increasing stress on the digestion process. Maximum efficiency of the grit removal facilities must be achieved to minimize the grit load entering the digesters. Although the draft tube design is superior to other types of mixers in maintaining sand suspension(5), most grit entering a digester will be deposited on the bottom and cannot be resuspended.

The frequency of cleaning varies depending upon the rate of accumulation of scum and grit which is in turn dependent on the efficiency of the grit removal process, the sludge loading rate, mixing efficiency, and the quality of solids received. Generally, cleaning must be done at intervals of three to eight years. One way to determine the need for cleaning is to probe the tank with rods or other devices through sample wells to determine the extent of the scum and grit accumulations. Comparing the usable digester volume with the volume required for the desired loading rate will also indicate the need for cleaning. Occasionally digesters require cleaning because draft tubes become clogged or partially clogged with rags and debris.

The digester tank bottom for Tank Nos. 7-12 is shown in Figure 12 and consists of four hoppers which are designed to facilitate cleaning. Operational experience in cleaning existing rectangular digesters with a variety of hopper designs led to the configuration shown and the following design features:

- Horizontal walkways are provided around the perimeter of the hoppers, allowing a safe and convenient work location for District personnel during cleanout operations.

- Cleanout and recirculation piping are installed in cleanout galleries under each digester, as shown schematically in Figure 14. The gallery provides complete access to all cleanout piping, which is subject to clogging during cleanout operations.

- High and low pressure primary effluent pipe connections are provided inside the digesters for washdown and dilution water purposes.

- Large openings, 2.44 m (8 ft) in diameter, are provided in the tank tops for personnel access to the tanks during cleaning, as shown in Figure 16. Access to the bottom is normally provided by either a specially designed ladder with safety cage, which is lowered by crane through the large access openings and secured to the walkway, or by a special type of moving platform which is air motor driven allowing personnel to enter the digesters safely and easily.

During tank cleanout, grit and other material are pumped from the hoppers by the cleanout pump through the drain piping shown in Figure 14. High pressure primary effluent is used to breakup the top scum layer as shown in Figure 15, while low pressure primary effluent is added to dilute the deposits to approximately 6 percent solids for pumping to the digester cleanings facility. High pressure primary effluent is also used to backflush the hopper drain lines when clogging occurs.

The cleanout pumps are connected by a force main to a central digester cleanings facility shown in Figure 1. This facility is shown schematically in Figure 15 and includes the following three stages:

- Stage 1 consists of self-cleaning, non-clogging sidehill screens called Hydrasieves. Larger solids in the flow stream are removed by the screens and conveyed to trucks for landfill disposal.

- Stage 2 utilizes cyclone separators with dewatering screws, which together are called Hydrogritters, to remove the grit from the flow stream. The grit removed is also conveyed to trucks for landfill disposal.

- Stage 3 utilizes dissolved air flotation with polymer addition to remove the remaining floatable and settleable material and a substantial portion of the suspended solids. Final effluent from the flotation system contains 200-300 mg/l of solids and is discharged to the inlet works head of JWPCP.

H. SAFETY FEATURES

As previously discussed, anaerobic digesters produce combustible methane gas which is explosive and contains hydrogen sulfide. Therefore, the following safety features have been incorporated into the design of the circular digestion system at JWPCP:

- Electric motors and lighting fixtures on the tank tops are explosion proof design.

- Explosion proof portable blowers are used to ventilate the digestion tanks during cleanout and provide 23.6 m^3/s (50,000 cfm) of fresh air.

- Pipe galleries shown in Figure 5 are continuously ventilated by a large positive ventilation system which is alarmed on failure.

I. MISCELLANEOUS DESIGN FEATURES

The following design features are in addition to those already discussed.

- The gas system at JWPCP consists of flow meters and condensate traps at each digester, a looped piping system around the plant, two waste gas burner stations, and metering of digester gas at all usage points. The waste gas burners are only needed when normal gas usage is interrupted.

- Each tank is equipped with nine sample wells as shown in Figure 16. A sample well consists of a seal pipe that extends below the sludge surface to prevent escape of digester gas when samples are withdrawn. A hookup to the compressed digester gas is provided to allow flushing the sample well before drawing a sludge sample.

- Each tank is equipped with four temperature wells and temperature sensing units, as shown in Figure 16. Centralized temperature indication is provided for all digestion tanks in the control building shown in Figure 5.

- Districts' field studies have shown that foaming in digesters receiving waste activated sludge should be minimal, providing that good mixing, uniform temperature and uniform feeding are maintained. However, temporary foaming may occur during startup of the digesters with waste activated sludge and during other periods of non-uniform operation. Should foaming occur, the level in the digesters can be lowered from the normal level of 1.2 m (4 ft) below the roof to a lower level of 2.4 m (8 ft) below the roof, as shown in Figure 13. This lower level will allow normal operation of the digestion system to continue during periods of temporary foaming while protecting against foam entering the gas system or gas recirculation compressors.

IV. DIGESTER CONTROL AND OPERATING EXPERIENCE

A. PROCESS CONTROLS

1. Temperature Control. The methane producing bacteria are particularly sensitive to changes in temperature. Rapid fluctuations in temperature can cause an imbalance between the acid and methane producing bacteria resulting in a digester upset. For mesophilic operation, the optimum temperature range is 30°C to 38°C (85°F to 100°F) with temperatures being maintained at about 35°C (95°F) in most installations. Satisfactory operation is possible anywhere within the range, but to allow time for the organisms to become acclimated, temperature changes greater than 0.5°C (1°F) per day should be avoided.

To achieve adequate control, digester temperatures should be monitored daily. Controls can be automatic, semi-automatic or manual. Where heating is done by external heat exchangers, a slight change in the setpoint is all that is needed to produce a minor temperature change. Manual control of direct steam injection is used at JWPCP. Operator experience is required to adjust the steam flow rate properly to produce the desired temperature change.

2. Mixing Control. The degree of control is dependent on the type of mixing device used. For mechanical mixers, control is limited to the number of hours operated per day and whether the upflow or downflow mode is used. Where gas mixing devices are utilized, the number of units in operation can be varied, as can the number of hours operated per day and the amount of gas injected per draft tube or injection point.

In addition to visual observation, which indicates only surface conditions, there are several methods to determine if mixing is adequate. One of the best methods is by measuring the temperature at various depths and at several locations in the tank. Any variation greater than 1 or 2°C (2 or 4°F) is indicative of poor mixing and possible future operational problems.

A similar analysis of mixing conditions can be made by taking samples at various locations and depths thoughout the tank and analyzing the samples for total solids or other components. This method is more time consuming, but it provides information on scum and grit deposits while the temperature profile does not.

A third method utilizes radioisotope or salt tracers to determine detention and mixing times. This is a relatively simple method but it can lead to false conclusions unless the volume of scum and grit deposits are known with some degree of accuracy.

3. Loading Control. For medium to large installations automated feed systems are normally provided. However, to insure satisfactory performance, a program of sampling, meter calibration and equipment maintenance is necessary. This program must involve all components of the system including the solids thickening facilities, incremental feed system and all measuring

and recording instruments. Failure to quickly observe and correct malfunctions can result in digester upsets. Frequent calculations must also be made to insure operation within the proper control ranges.

B. ANALYTICAL CONTROLS

Figures 17 and 18 graphically display the results of analytical tests performed to determine the conditions and efficiency of the digestion systems at JWPCP. Value and use of each type of test is described briefly below.

1. pH. Daily determinations of pH are considered normal practice in digester operation. Due to the normal buffering capacity, the pH changes quite slowly and thus it is not an indicator of digester operation to be relied on by itself.

2. Alkalinity. This test measures the relative buffering capacity of the system. It too is a relatively insensitive indicator, even when used in conjunction with pH measurements.

3. Volatile Acids. The volatile acids content of the digesting sludge is probably the most universally used indicator of digester condition. Proper operating levels of the volatile acids will depend on sludge feed characteristics, detention time and other operational variables. There are a variety of methods used for determining the volatile acid content. Each method has its own range to be used as a guideline for successful operation. All fall within the range of 50-150 mg/l with an upper limit for any system of approximately 800 mg/l.

4. Volatile Acid-Alkalinity Relationship. The ratio of volatile acids to alkalinity and particularly the rate of change in this relationship is an excellent means of determining the digester environment. Each plant will have its own characteristic ratio for proper operation. Generally the ratio will be less than 0.1 and any increase above 0.3 to 0.4 indicates severe stress and the need to take immediate corrective action. As the ratio approaches 0.8, there is a depression in pH indicating the digester is then in serious trouble. Remedial action at this time may be too late to stop a major decline in digester efficiency.

5. Gas Production. Digester gas usually has a composition of 35-40 percent carbon dioxide and 60-65 percent methane with trace amounts of hydrogen sulfide and other gases. The heat value may range from 18,600 - 26,000 kJ/m^3 (500-700 BTU/cu ft) depending on the type of waste treated and degree of digestion accomplished.

Both the quantity and quality of the digester gas are indicators of digester conditions. Any sharp drop in gas production , as measured by individual gas meters, is cause for alarm and indicative of potential problems. Under normal conditions the gas produced will have a fairly uniform composition of methane and carbon dioxide. Any deviation from this norm, such as an increase in the carbon dioxide content, is a warning of impending trouble and indicates the need for investigation.

Calculations should be performed routinely to determine the amount of gas produced per unit of volatile matter destroyed. Expected production ranges from 750 l/kg (12 cu ft/lb) to 1,125 l/kg (18 cu ft/lb), depending on the sludge characteristics and operational conditions.

Fats and greases yield the highest gas production, carbohydrates fall in the middle, and proteins produce the least quantity of gas. A long range decline in gas production per unit of volatile matter destroyed generally indicates a change in sludge characteristics.

6. Volatile Solids Destruction. Volatile solids destructin is a measure of the amount of organic matter converted to the end products of water and gas. It is not a tool for daily use in determining digester efficiency, but rather for determining loadings and long-term efficiency. The percent reduction of volatile matter is dependent on the sludge characteristics, detention time, and digester operation. For primary sludges, a volatile solids reduction of 50-60 percent should be attainable. Due to the fact that secondary sludges are more resistant to digestion, somewhat lower volatile destruction results, as shown in Table 1.

C. PHYSICAL CONTROLS

1. Loadings. For any large system, the analytical tests chosen for control of digester operation should be performed at least five days each week. When the results of the tests are available, the conditions of each digester should be assessed and the loadings for the next day should be determined. For any digester that shows even minimal signs of stress, the loading must be reduced slightly. Conversely, for digesters which show normal or healthy conditions, the loadings may be gradually increased. If sufficient capacity exists, the loadings can be adjusted from digester to digester on a daily basis while maintaining good digestion efficiency at very high loading rates.

2. Toxic Wastes. Frequently toxicity is blamed for digester upsets, but only rarely is it found to be the causative agent, particularly in large treatment plants. Heavy metals are usually the prime suspect. However, normal digester operation produces sufficient hydrogen sulfide which forms insoluble compounds with the heavy metals making them unavailable to the organisms. As long as hydrogen sulfide is present in the digester gas, heavy metal toxicity should not be suspected. Strong organic bactericides, pesticides and fungicides can be present in high enough concentrations to cause toxicity. Cases of this type are normally associated only with small systems with a large contribution from industries producing these types of chemicals. Strong oxidants, such as nitrates and chromates, are considered to be toxicants; but due to the septicity of digesting sludge, they must be present in very high concentrations to cause trouble. Ammonium, potassium, sodium, magnesium, and calcium ions also can be toxic if chemical agents are used for pH control, particularly if ammonia or sodium componds are used. Care should be taken to exclude high concentrations of known toxicants from the collection system by adequate source control.

V. COSTS

A. CONSTRUCTION

Construction of Digestion Tanks 7-12, which complete the master plan as outlined previously, began in September 1979 and will be completed in three years. These tanks are being constructed at a cost of $17,500,000, which corresponds to $210 per m^3 ($5.80 per cu ft) of digester capacity.

B. OPERATION AND MAINTENANCE

Table 2 presents operation and maintenance costs for digester operation at JWPCP during 1979. Included is their relationship to total plant costs, as well as unit costs per cubic meter (million gallons) of flow and cost per metric ton (ton) of dry solids digested. Also shown is the amount of gas production and its relative value compared to an equivalent amount of natural gas at current prices. It is readily apparent from Table 2 that the value of the gas produced is over 2.8 times the operation and maintenance costs associated with its production.

VI. SUMMARY

1. The efficient and complete utilization of the digester gas at JWPCP will provide almost 80 percent of the total energy requirements for the entire plant after completion of the secondary treatment expansion.
2. The Sanitation Districts have achieved dependable operation and maximum methane gas production from anaerobic sludge digestion by providing complete mixing, uniform mesophilic temperatures, and uniform sludge feeding. The design and construction of anaerobic digesters which can be operated at high organic loading rates minimizes digester volume while allowing a constant supply of digester gas to be produced for energy recovery.
3. The Sanitation Districts' circular digestion system at JWPCP has proven to be an economical design based on construction cost, land utilization, operating costs, and operational flexibility.

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TABLE 1. Design Criteria for Los Angeles County Sanitation Districts Circular Digestion System at JWPCP.

Digestion Tanks	
Tank Configuration	Circular
Number of Tanks	12
Diameter per Tank	38 m (125 ft)
Volume per Tank	14,200 m^3 (500,000 cu ft)
Side Water Depth	11m (36 ft)
Maximum Water Depth	15.2 m (50 ft)
Data for Digestion of 100% Waste Activated Sludge	
Detention Times (6 tanks in service)	
At Average Sludge Flow	21 days
At Maximum Sludge Flow	14 days
Volatile Solids Loading @ Average Flow	1.3 $kg/m^3/d$ (0.08 lb/cu ft/d)
Volatile Solids Destruction	30%
Data for Digestion of 100% Primary Sludge or Combined Digestion	
Detention Times	
At Average Sludge Flow	15 days
At Maximum Sludge Flow	10 days
Volatile Solids Loading	
At Average Sludge Flow	2.4 $kg/m^3/d$ (0.15 lb/cu ft/d)
Volatile Solids Destruction	50%
Digester Mixing System	
Method	Gas Recirculation w/Internal Draft Tubes
Number of Gas Mixers per Tank	5
Type of Gas Compressors	Rotary Positive Displacement
Number of Gas Compressors per Tank	2
Capacity per Gas Compressor	1.4 m^3/s (2900 cfm)
Discharge Pressure per Gas Compressor	52 kN/m^3 (7.5 psig)
Number of Gas Compressors Operated (maximum)	1
Digester Heating System	
Method	Direct Steam Injection
Number of Boilers	3
Number of Boilers Operated (maximum)	2
Rating of Boiler (each)	370 kW (500 hp)
Capacity of Boiler (each)	7800 kg/h (17,500 lb/hr)
Boiler Fuel	Digester Gas
Digester Temperature	35°C (95°F)
Digester Feeding System	
Method	Incremental Volumetric Feeding
Number of Incremental Feedings per Day	10
Control System	Automatic Incremental Feed Counters and Valve Sequence Controls
Digester Cleanout System	
Number of Cleanout Pump Stations	6
Type of Pumps	Horizontal Centrifugal w/V-belt Drives
Number of Pumps for Two Tanks	1
Capacity per Pump	0.095 m3/s (1500 gpm)

TABLE 2. Summary of 1979 Operation & Maintenance Costs for Anaerobic Digestion at JWPCP.

	Jan-June		July-Dec	
Total Digester Costs - $	692,480		719,755	
% of Total Plant Costs	13.50		14.00	
Cost $\$10^3/m^3$ (\$/MG) of Flow	2.75	(10.40)	2.85	(10.70)
Metric Tons (Tons) - dry wt.	88,300	(97,300)	91,800	(101,200)
Cost $/Metric Ton ($/Ton) - dry wt.	7.85	(7.10)	7.85	(7.10)
Gas Produced - 10^6 m^3 (10^6 cu ft)	31	(1,080)	29	(1,029)
10^6 kJ (10^6 BTU) Produced	736,000	(698,000)	711,000	(674,000)
Natural Gas Value Equivalent @ \$2.85/10^6 kJ (\$3/10^6 BTU) - $	2,098,000		2,026,000	

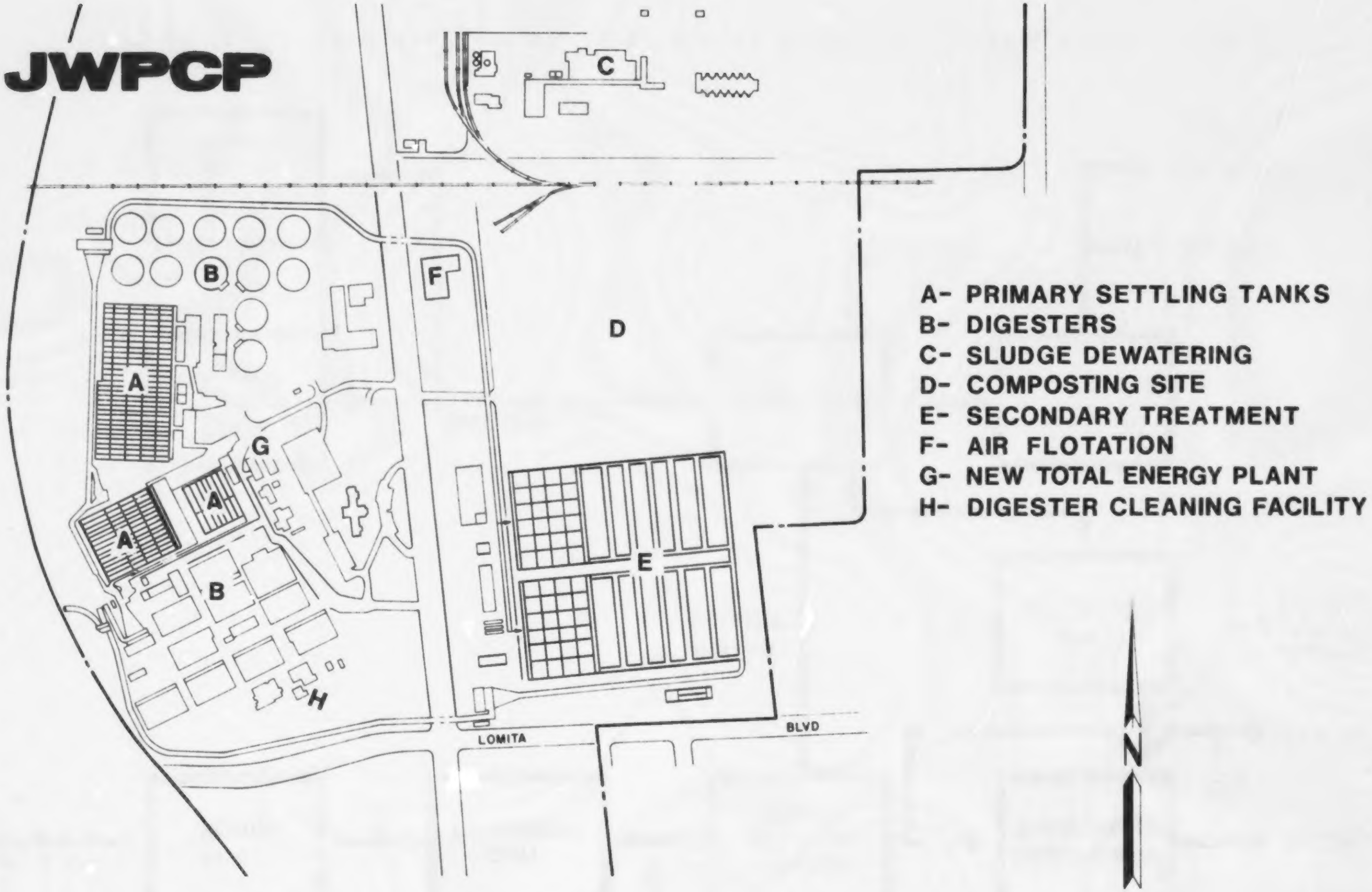
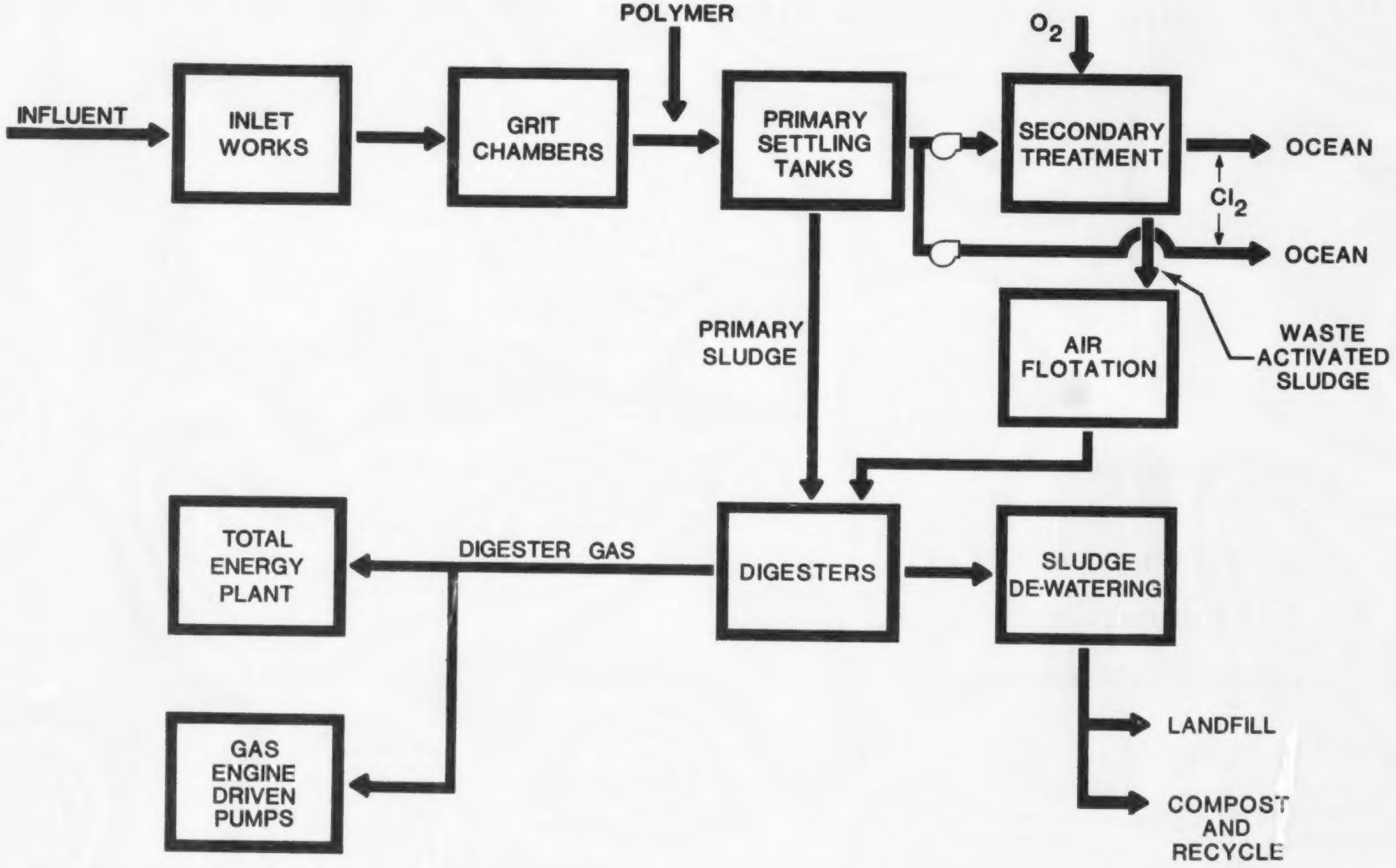

JWPCP
A- PRIMARY SETTLING TANKS
B- DIGESTERS
C- SLUDGE DEWATERING
D- COMPOSTING SITE
E- SECONDARY TREATMENT
F- AIR FLOTATION
G- NEW TOTAL ENERGY PLANT
H- DIGESTER CLEANING FACILITY
A
A
A
B
B
C
D
E
F
G
H
LOMITA
BLVD
N

FIGURE 1. Site layout of Joint Water Pollution Control Plant.



INFLUENT
INLET WORKS
GRIT CHAMBERS
POLYMER
PRIMARY SETTLING TANKS
O_2
SECONDARY TREATMENT
OCEAN
Cl_2
OCEAN
WASTE ACTIVATED SLUDGE
AIR FLOTATION
PRIMARY SLUDGE
DIGESTERS
DIGESTER GAS
TOTAL ENERGY PLANT
GAS ENGINE DRIVEN PUMPS
SLUDGE DE-WATERING
LANDFILL
COMPOST AND RECYCLE

FIGURE 2. Flow diagram of Joint Water Pollution Control Plant (1983).

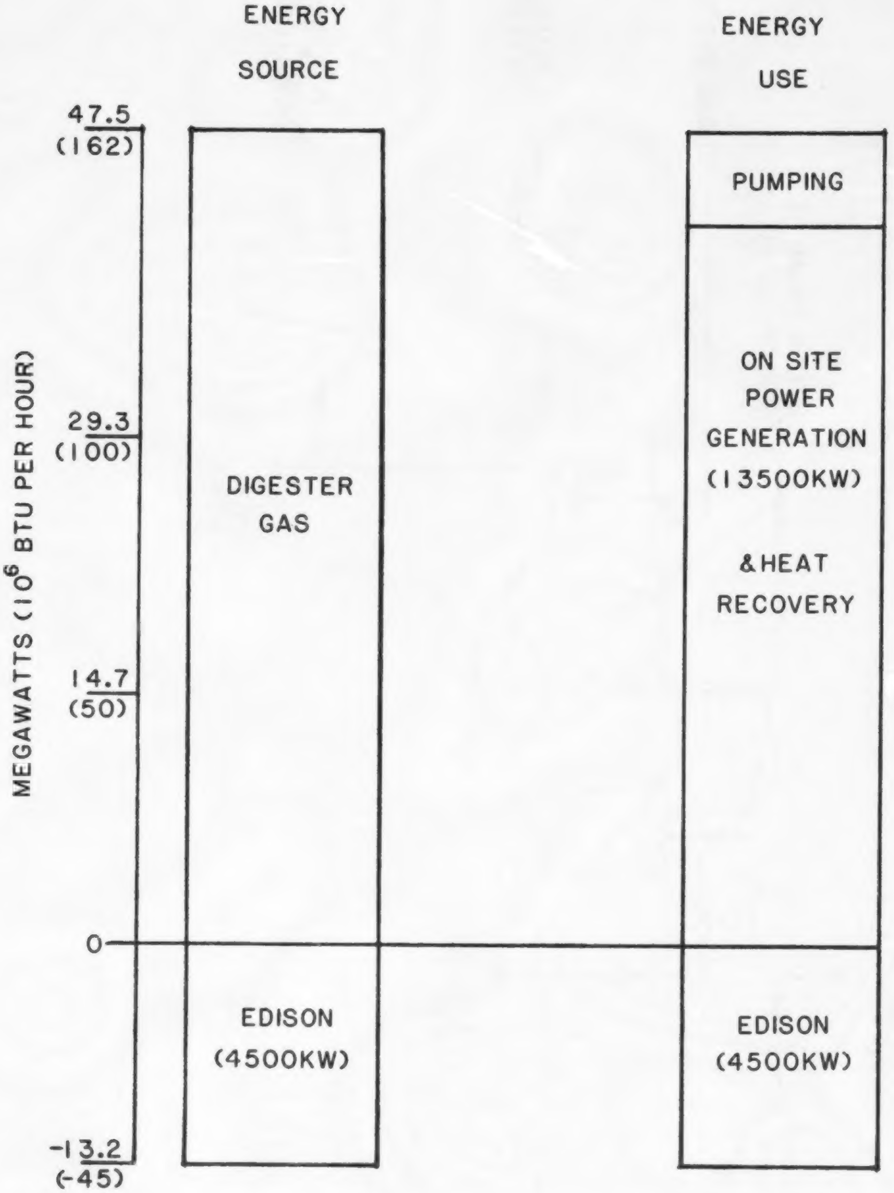


FIGURE 3. 1983 JWPCP Energy Balance.

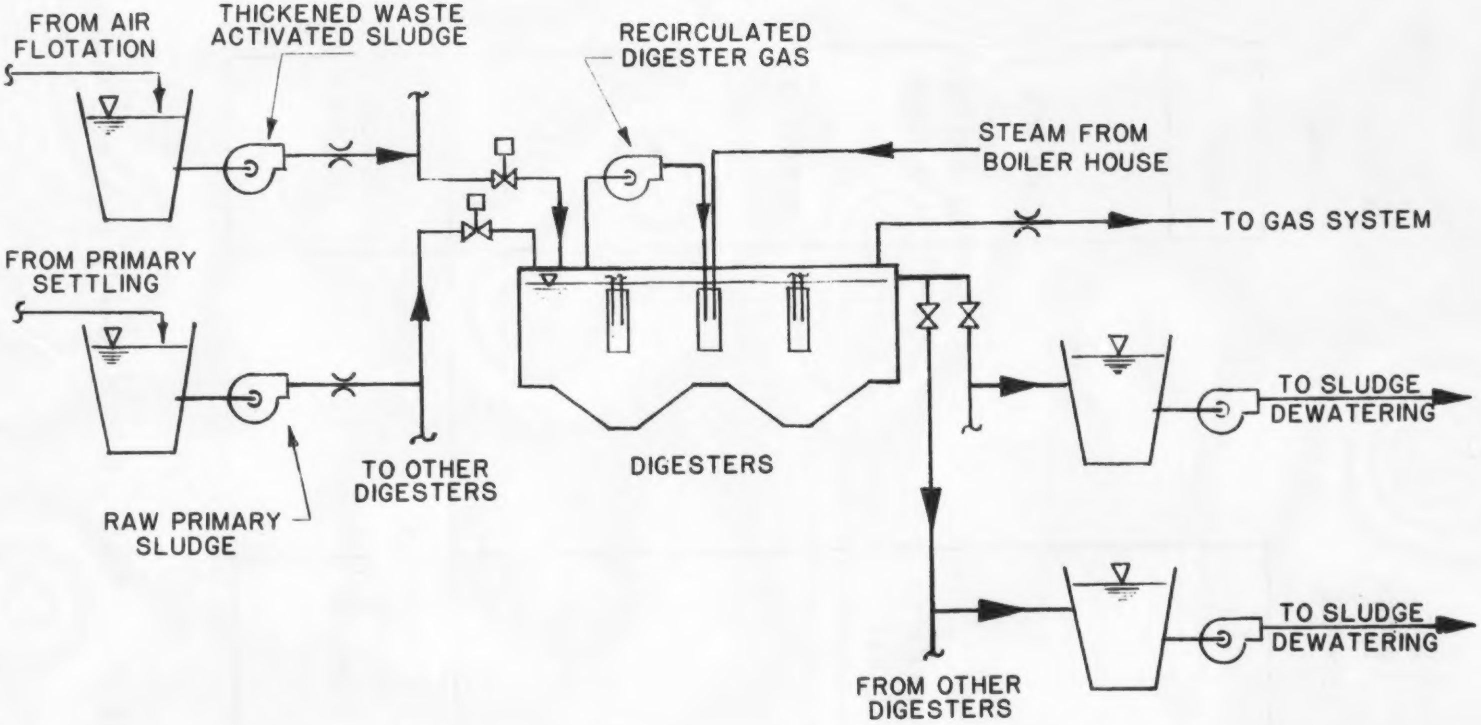

FROM AIR FLOTATION
THICKENED WASTE ACTIVATED SLUDGE
RECIRCULATED DIGESTER GAS
STEAM FROM BOILER HOUSE
TO GAS SYSTEM
FROM PRIMARY SETTLING
TO SLUDGE DEWATERING
RAW PRIMARY SLUDGE
TO OTHER DIGESTERS
DIGESTERS
TO SLUDGE DEWATERING
FROM OTHER DIGESTERS

FIGURE 4. Flow schematic of Circular Digestion System at JWPCP.

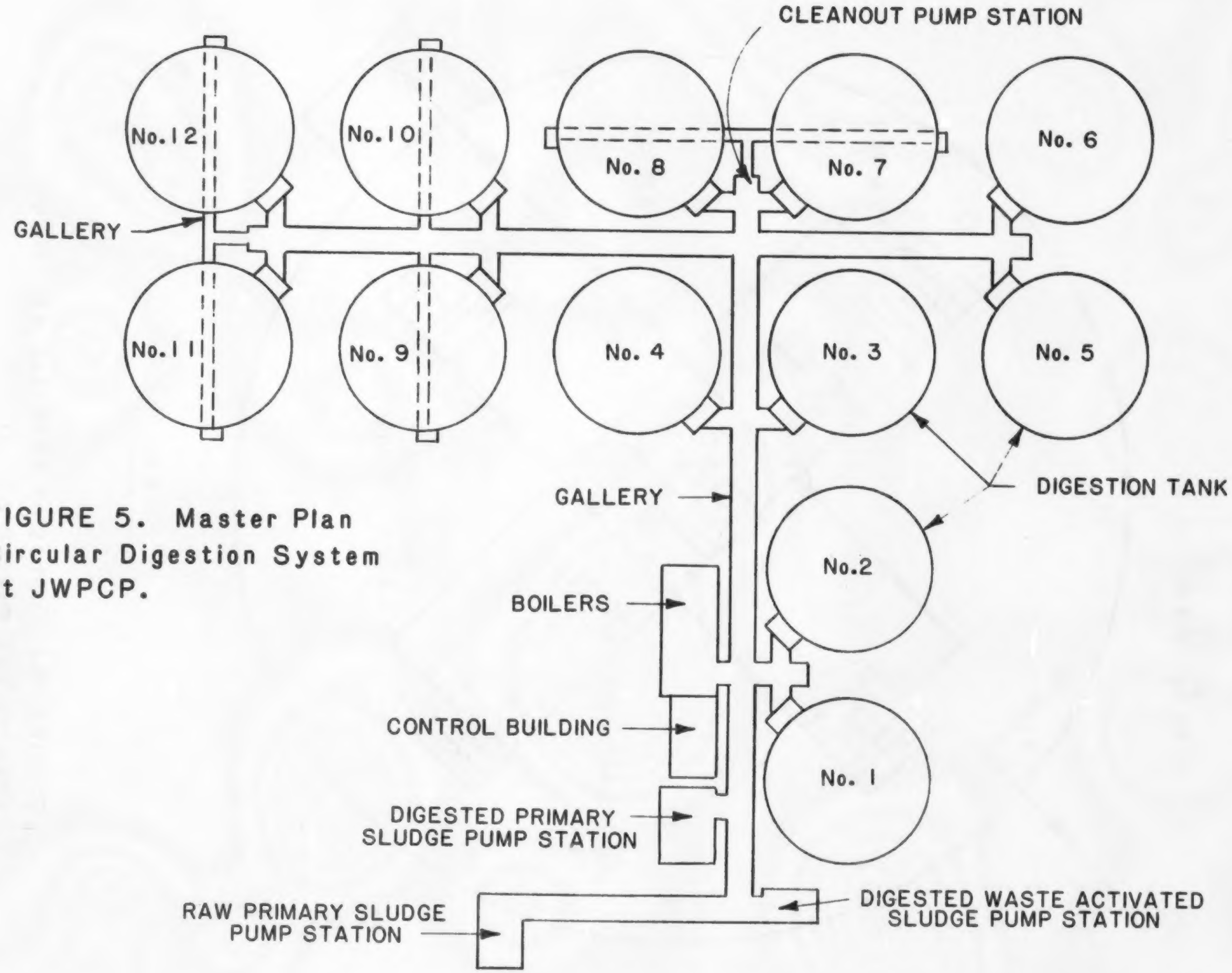
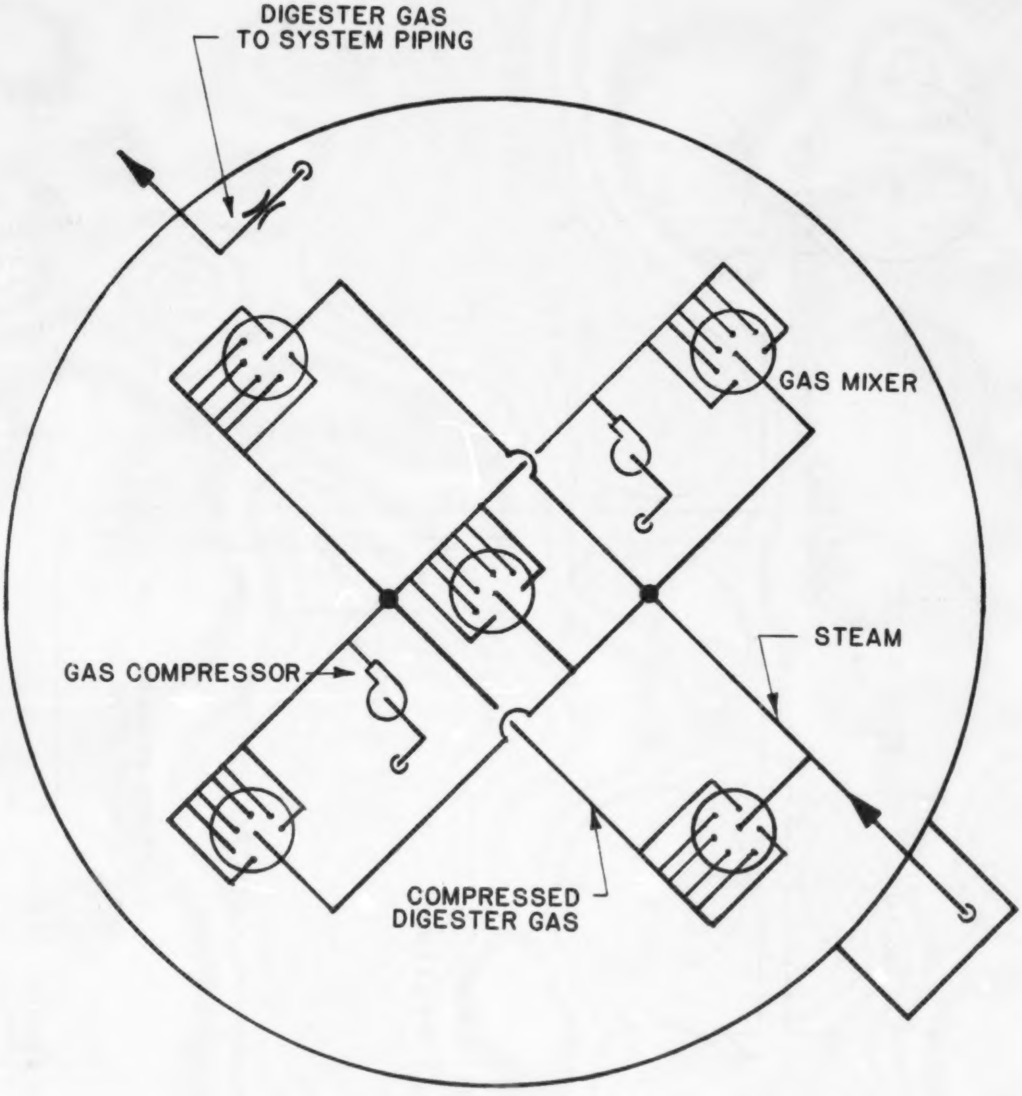

CLEANOUT PUMP STATION
No.12
No.10
No. 8
No. 7
No. 6
GALLERY
No.11
No. 9
No. 4
No. 3
No. 5
GALLERY
DIGESTION TANK
No.2
BOILERS
CONTROL BUILDING
No. 1
DIGESTED PRIMARY SLUDGE PUMP STATION
RAW PRIMARY SLUDGE PUMP STATION
DIGESTED WASTE ACTIVATED SLUDGE PUMP STATION

FIGURE 5. Master Plan Circular Digestion System at JWPCP.


DIGESTER GAS
TO SYSTEM PIPING
GAS MIXER
STEAM
GAS COMPRESSOR
COMPRESSED
DIGESTER GAS

PLAN

FIGURE 6. Digestion tank top at JWPCP showing gas mixers and piping.

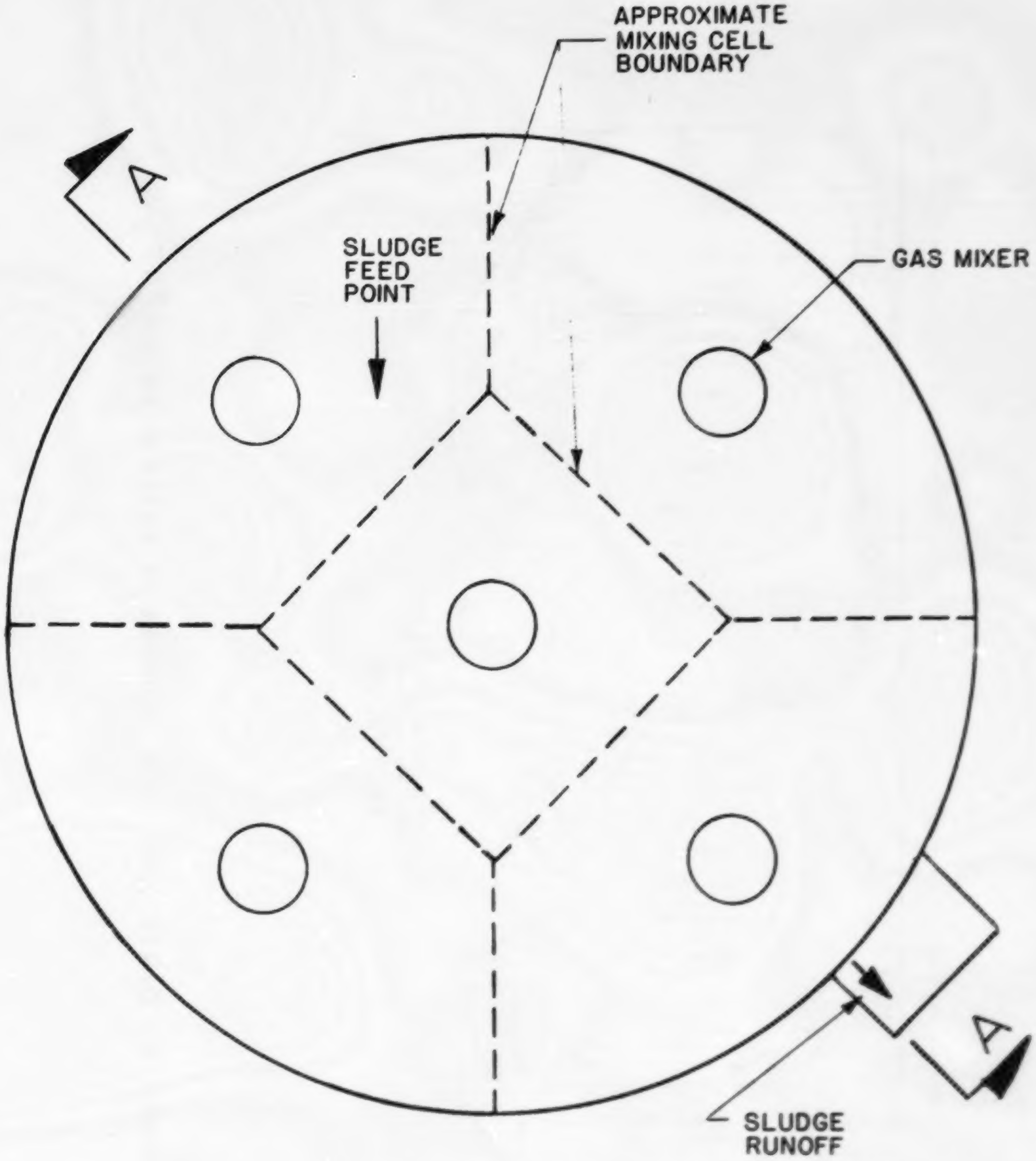
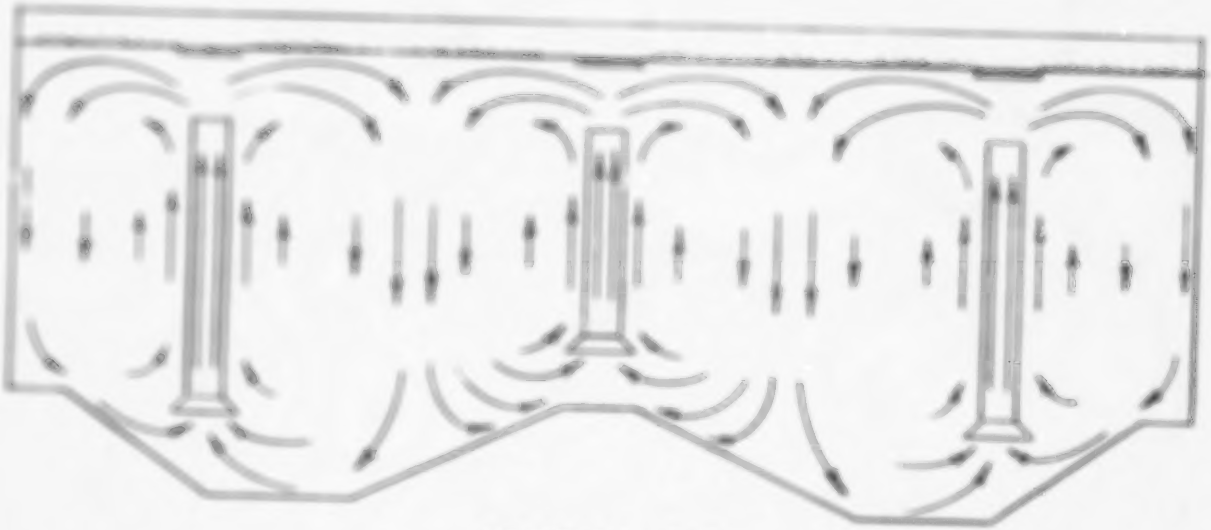

APPROXIMATE
MIXING CELL
BOUNDARY
A
SLUDGE
FEED
POINT
GAS MIXER
A
SLUDGE
RUNOFF
PLAN

FIGURE 7. Digestion tank top at JWPCP showing mixing cell configuration



SECTION A-A
(SEE FIGURE 7)

FIGURE 8. Draft tube flow circulation pattern at JWPCP

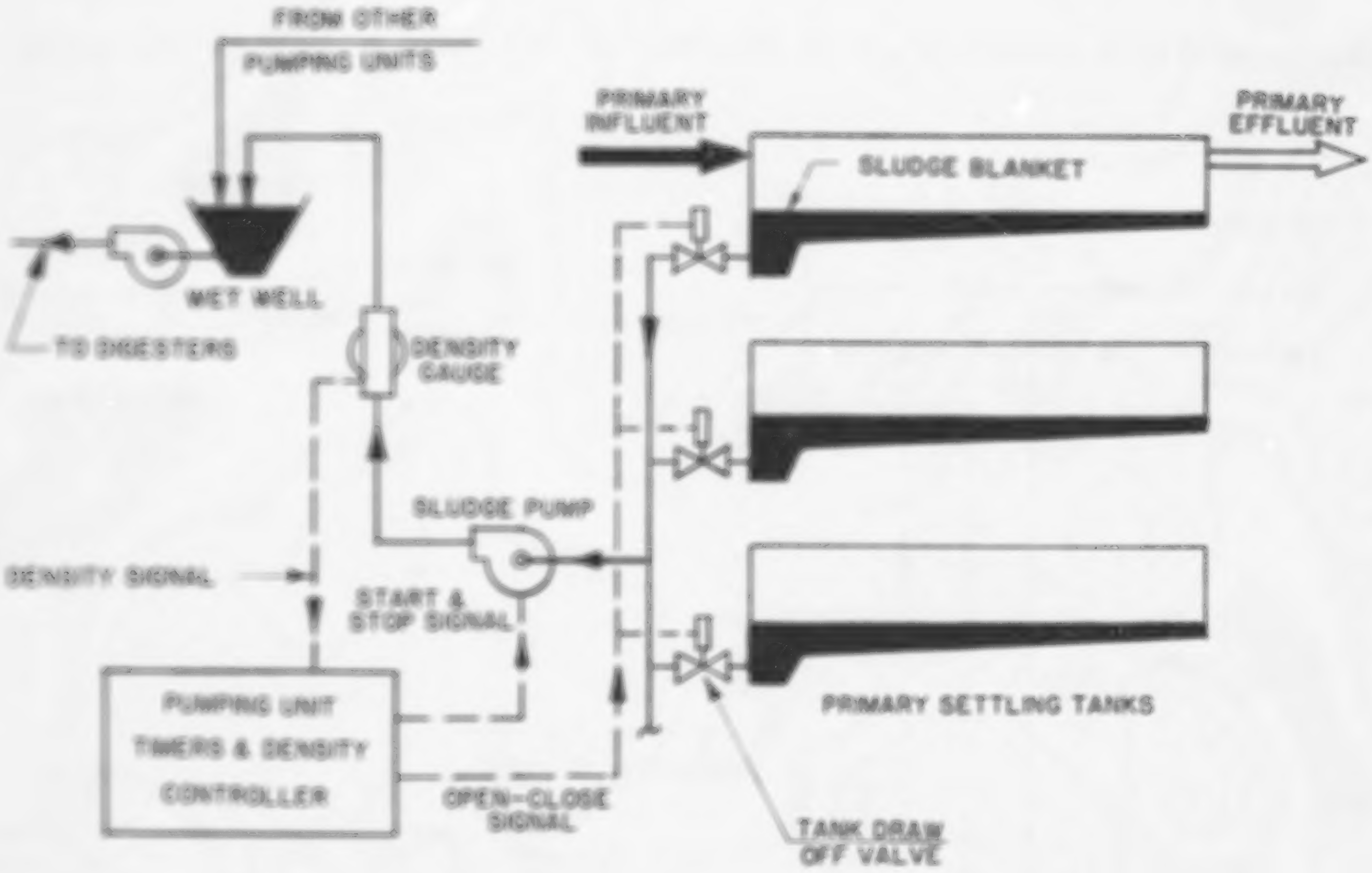

FROM OTHER
PUMPING UNITS
PRIMARY INFLUENT
PRIMARY EFFLUENT
SLUDGE BLANKET
WET WELL
TO DIGESTERS
DENSITY GAUGE
SLUDGE PUMP
DENSITY SIGNAL
START & STOP SIGNAL
PUMPING UNIT TIMERS & DENSITY CONTROLLER
OPEN-CLOSE SIGNAL
PRIMARY SETTLING TANKS
TANK DRAW OFF VALVE

FIGURE 9. Schematic of a raw sludge pumping unit at JWPCP

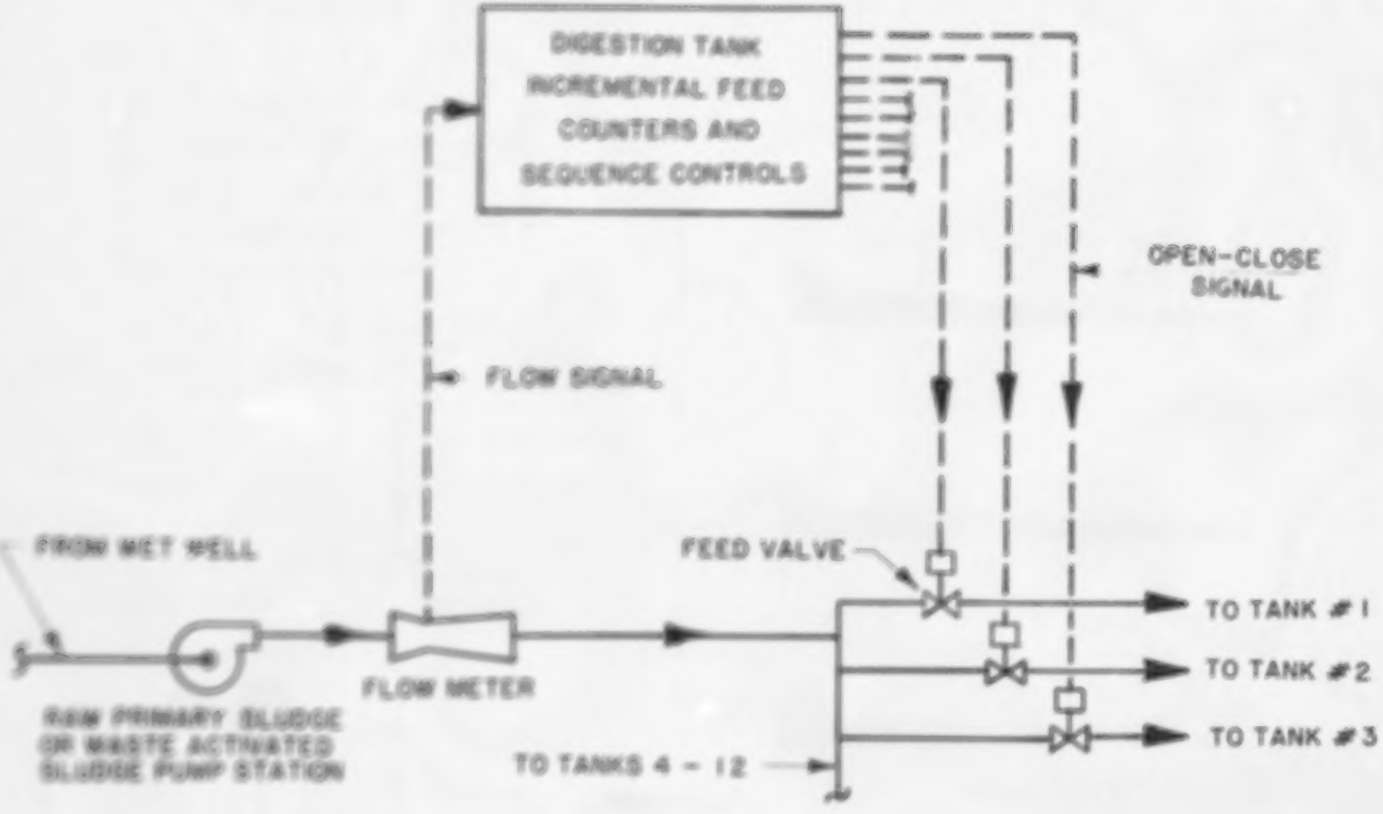

DIGESTION TANK
INCREMENTAL FEED
COUNTERS AND
SEQUENCE CONTROLS
OPEN-CLOSE
SIGNAL
FLOW SIGNAL
FROM WET WELL
FEED VALVE
FLOW METER
RAW PRIMARY SLUDGE
OR WASTE ACTIVATED
SLUDGE PUMP STATION
TO TANKS 4 - 12
TO TANK # 1
TO TANK # 2
TO TANK # 3

FIGURE 10. Schematic of the circular digestion tank feed control system at JWPCP

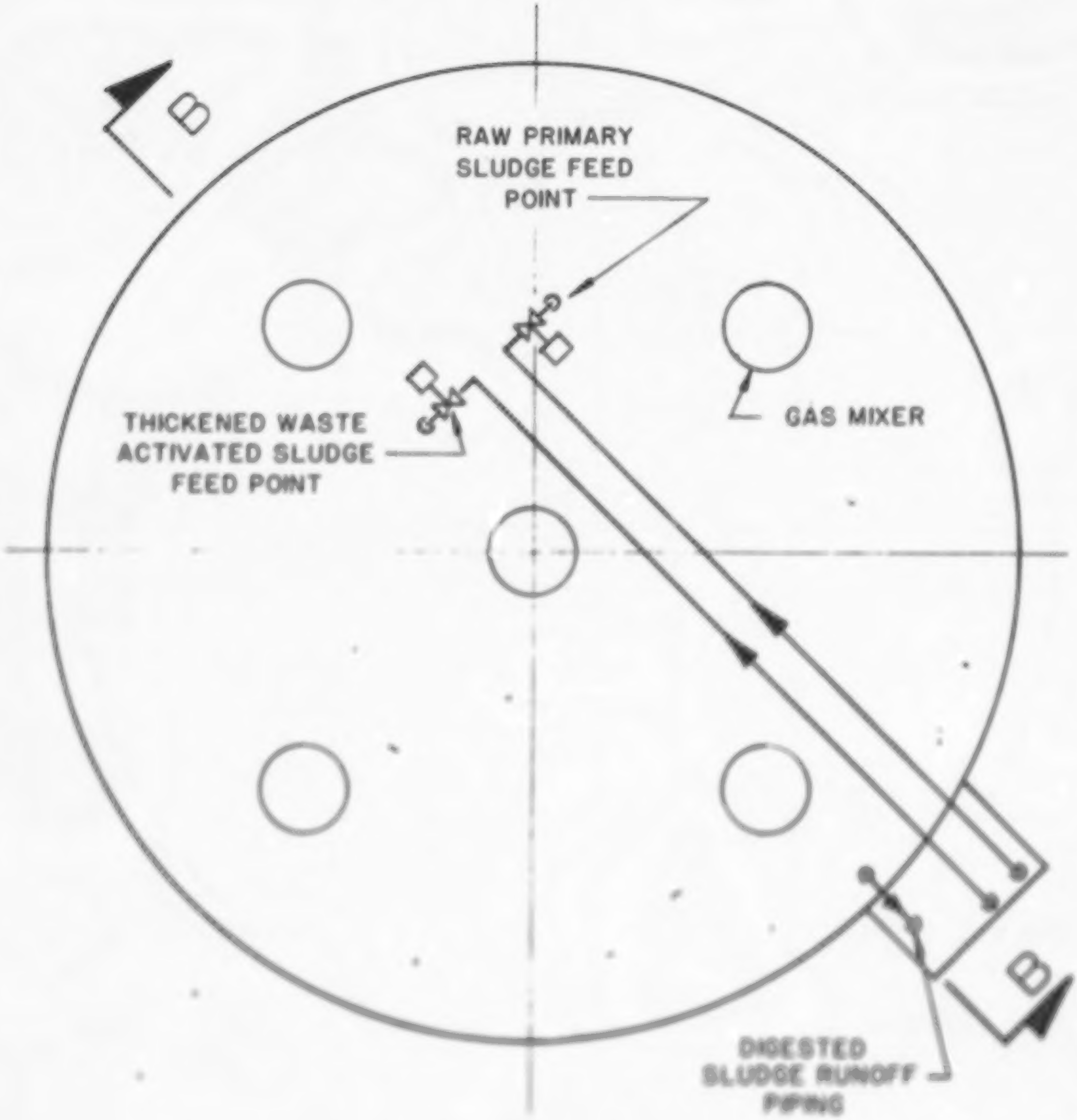
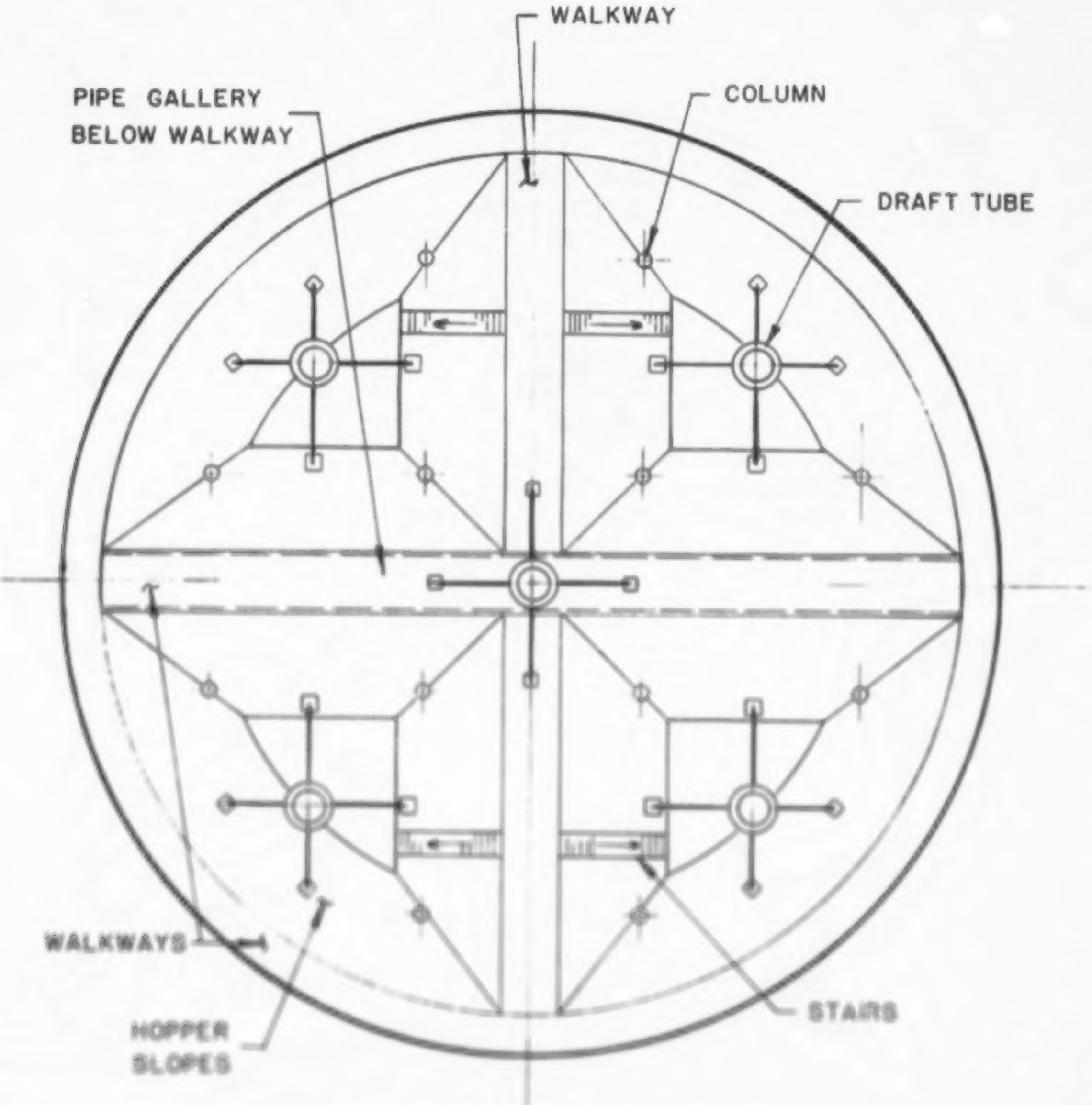
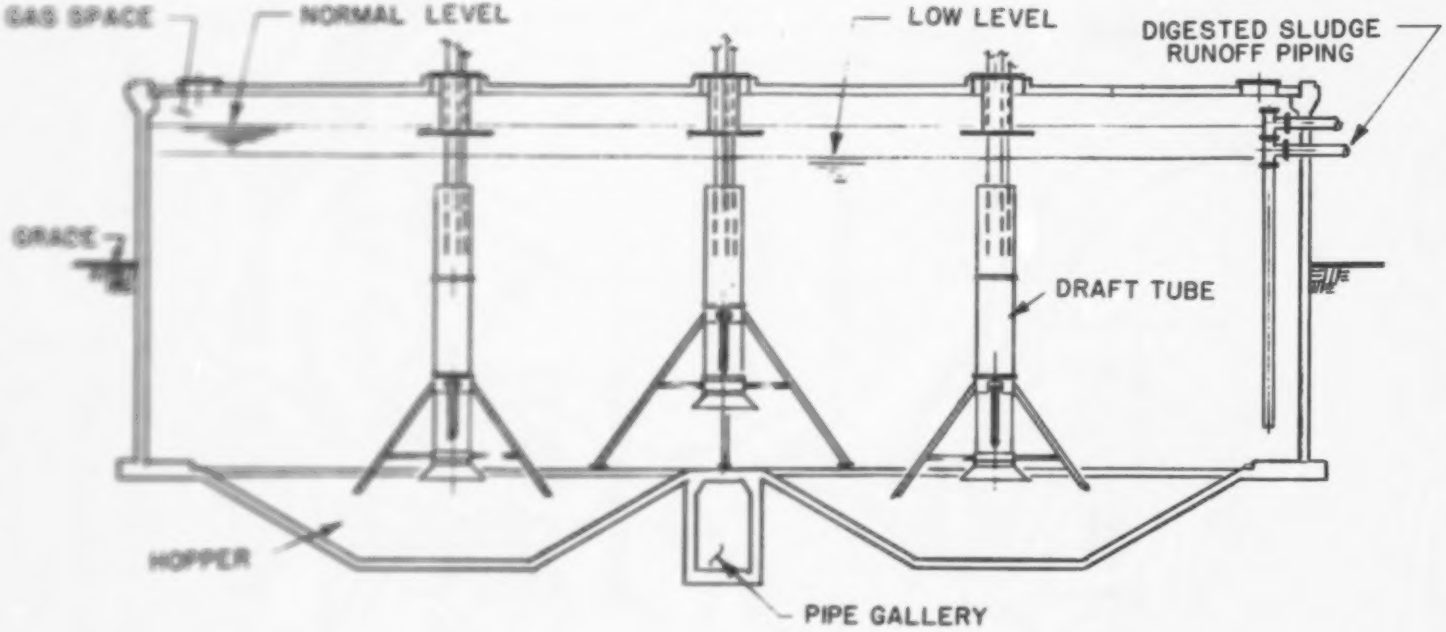

B
RAW PRIMARY
SLUDGE FEED
POINT
GAS MIXER
THICKENED WASTE
ACTIVATED SLUDGE
FEED POINT
B
DIGESTED
SLUDGE RUNOFF
PIPING

FIGURE 11. Digestion tank top at JWPCP showing sludge feed and runoff piping


WALKWAY
PIPE GALLERY
BELOW WALKWAY
COLUMN
DRAFT TUBE
WALKWAYS
HOPPER
SLOPES
STAIRS

PLAN SECTION

FIGURE 12. Digestion tank hopper bottom at JWPCP


GAS SPACE
NORMAL LEVEL
LOW LEVEL
DIGESTED SLUDGE RUNOFF PIPING
GRADE
DRAFT TUBE
HOPPER
PIPE GALLERY

SECTION B-B
(SEE FIGURE 11)

FIGURE 13. Digestion tank cross section at JWPCP

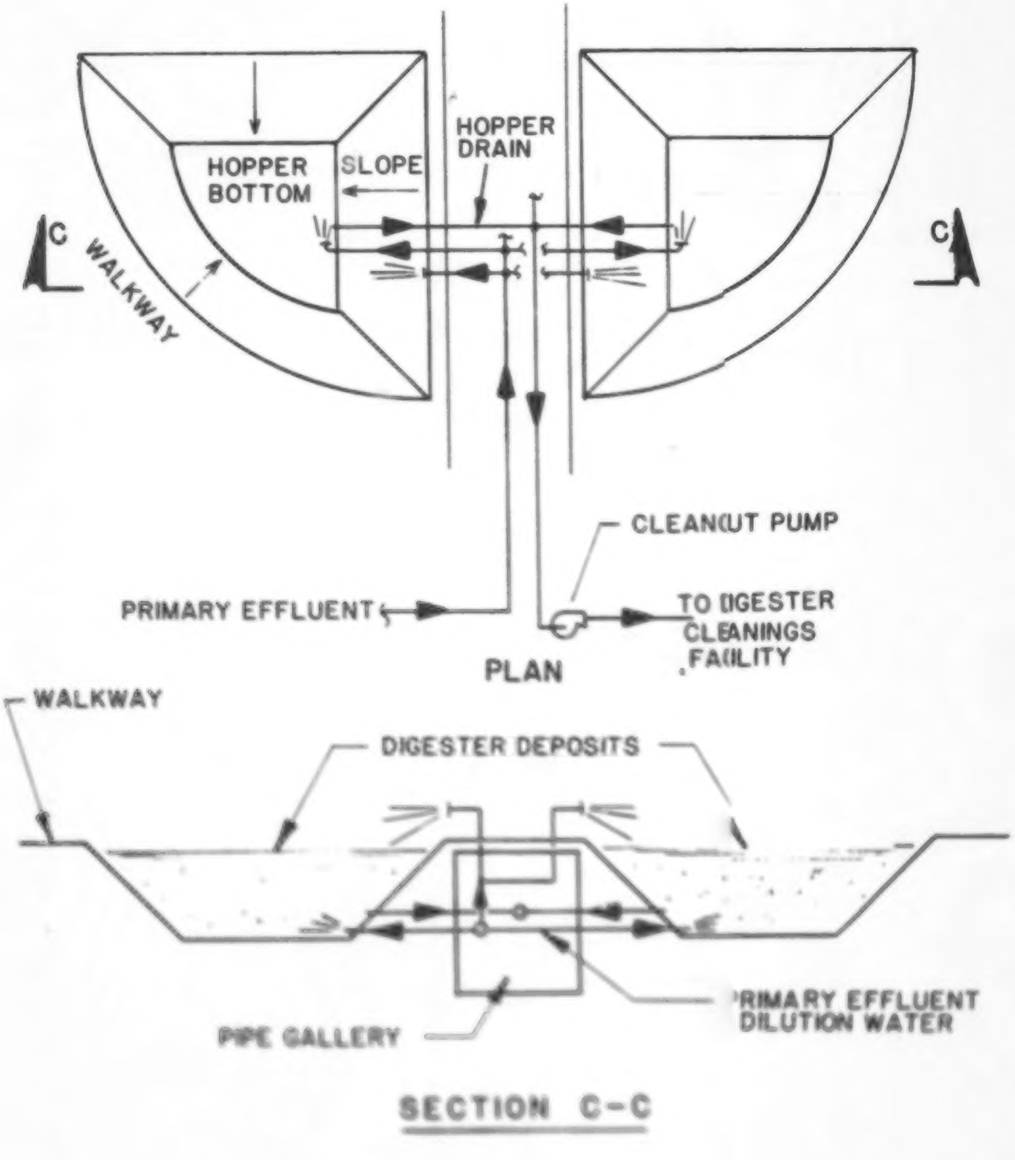

HOPPER BOTTOM
SLOPE
HOPPER DRAIN
C
C
WALKWAY
CLEANOUT PUMP
PRIMARY EFFLUENT
TO DIGESTER CLEANINGS FACILITY
PLAN
WALKWAY
DIGESTER DEPOSITS
PRIMARY EFFLUENT DILUTION WATER
PIPE GALLERY
SECTION C-C

FIGURE 14. Circular digestion tank hoppr and cleanout gallery piping at JWPCP

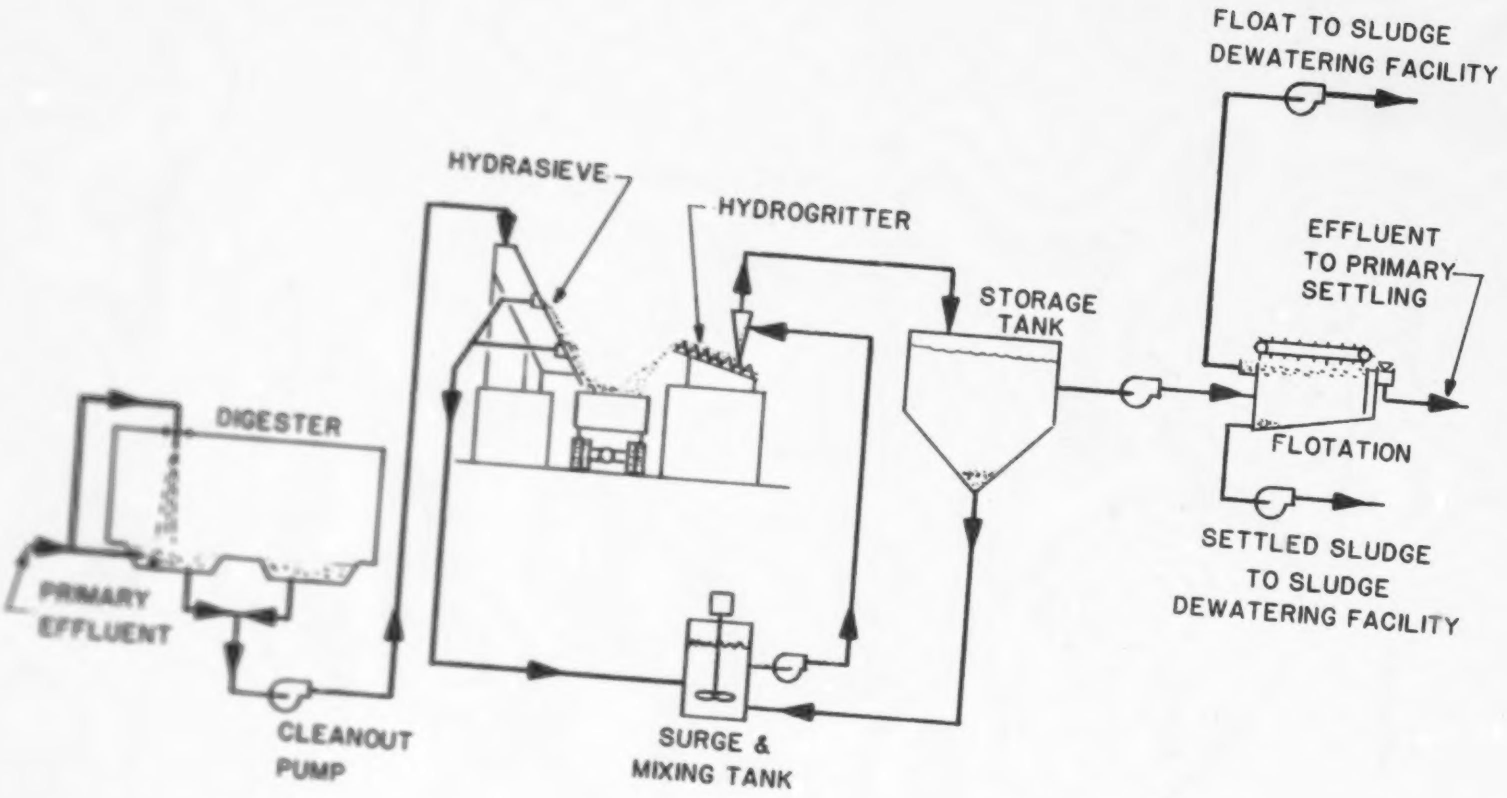

FLOAT TO SLUDGE
DEWATERING FACILITY
HYDRASIEVE
HYDROGRITTER
EFFLUENT
TO PRIMARY
SETTLING
STORAGE
TANK
DIGESTER
FLOTATION
PRIMARY
EFFLUENT
SETTLED SLUDGE
TO SLUDGE
DEWATERING FACILITY
CLEANOUT
PUMP
SURGE &
MIXING TANK

FIGURE 15. Flow schematic of Digester Cleanings Facility at JWPCP

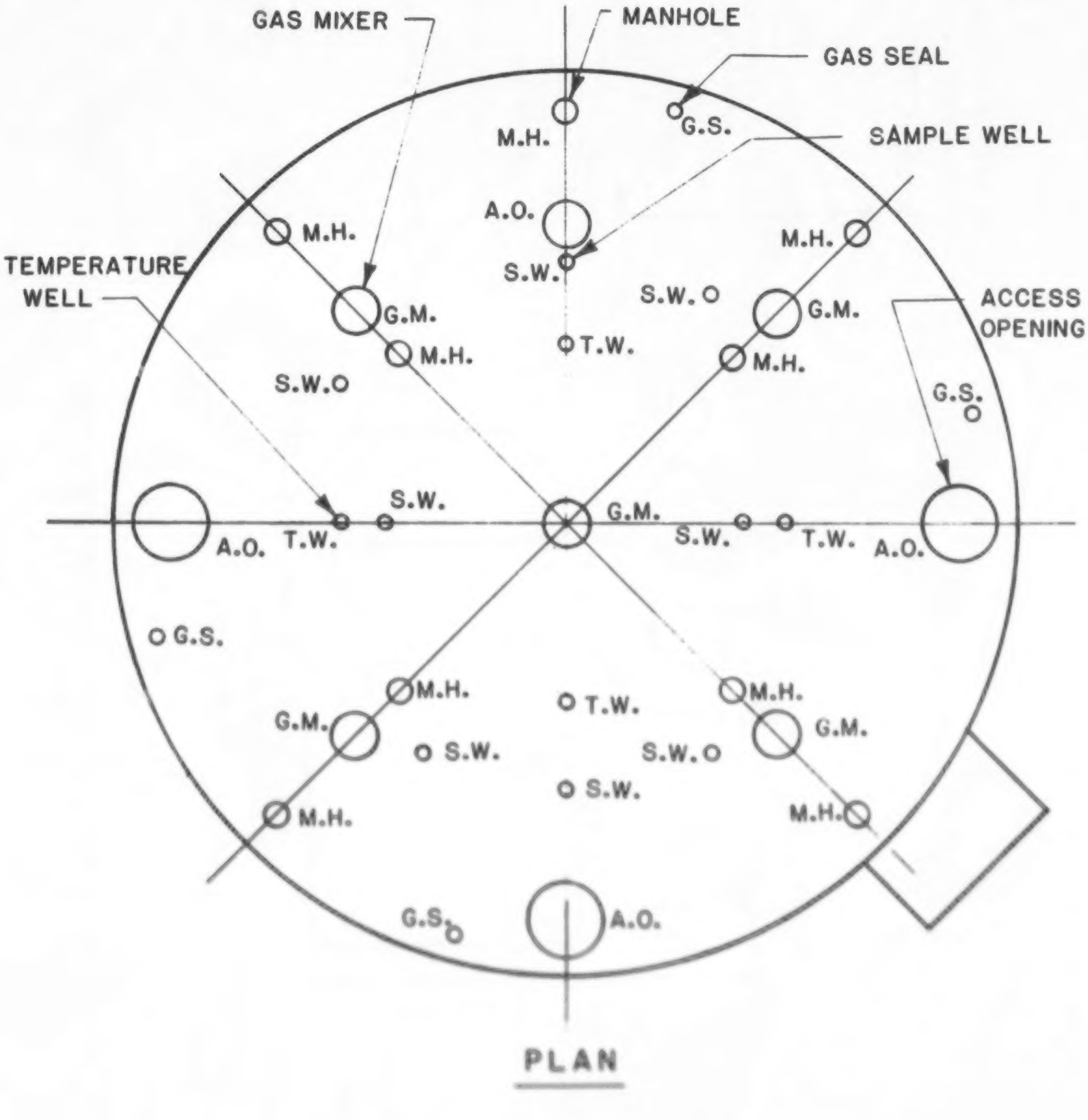

GAS MIXER
MANHOLE
GAS SEAL
M.H.
G.S.
SAMPLE WELL
A.O.
M.H.
M.H.
S.W.
TEMPERATURE WELL
S.W.
G.M.
G.M.
ACCESS OPENING
M.H.
T.W.
M.H.
S.W.
G.S.
S.W.
G.M.
A.O.
T.W.
S.W.
T.W.
A.O.
G.S.
M.H.
M.H.
T.W.
G.M.
G.M.
S.W.
S.W.
S.W.
M.H.
M.H.
G.S.
A.O.
PLAN

FIGURE 16. Digestion tank top at JWPCP showing access openings and penetrations

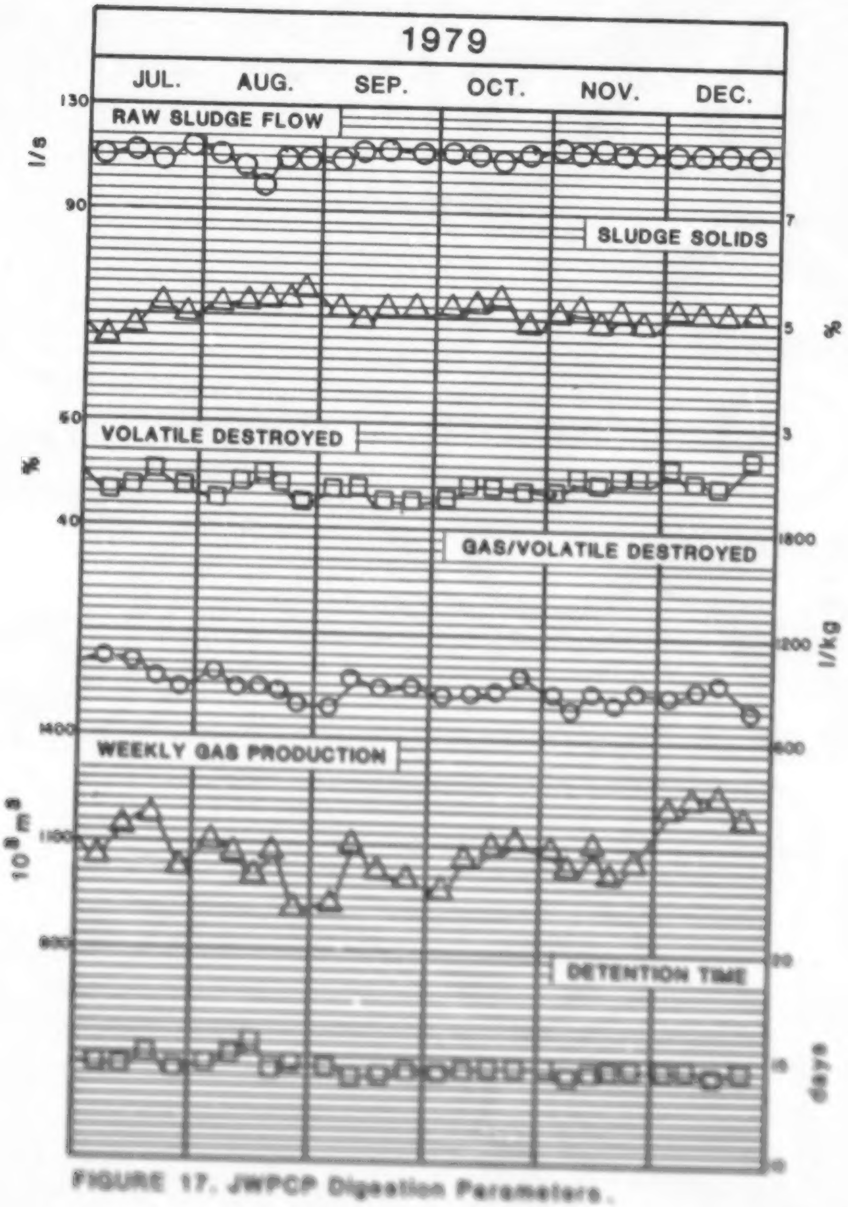


FIGURE 17. JWPCP Digestion Parameters.

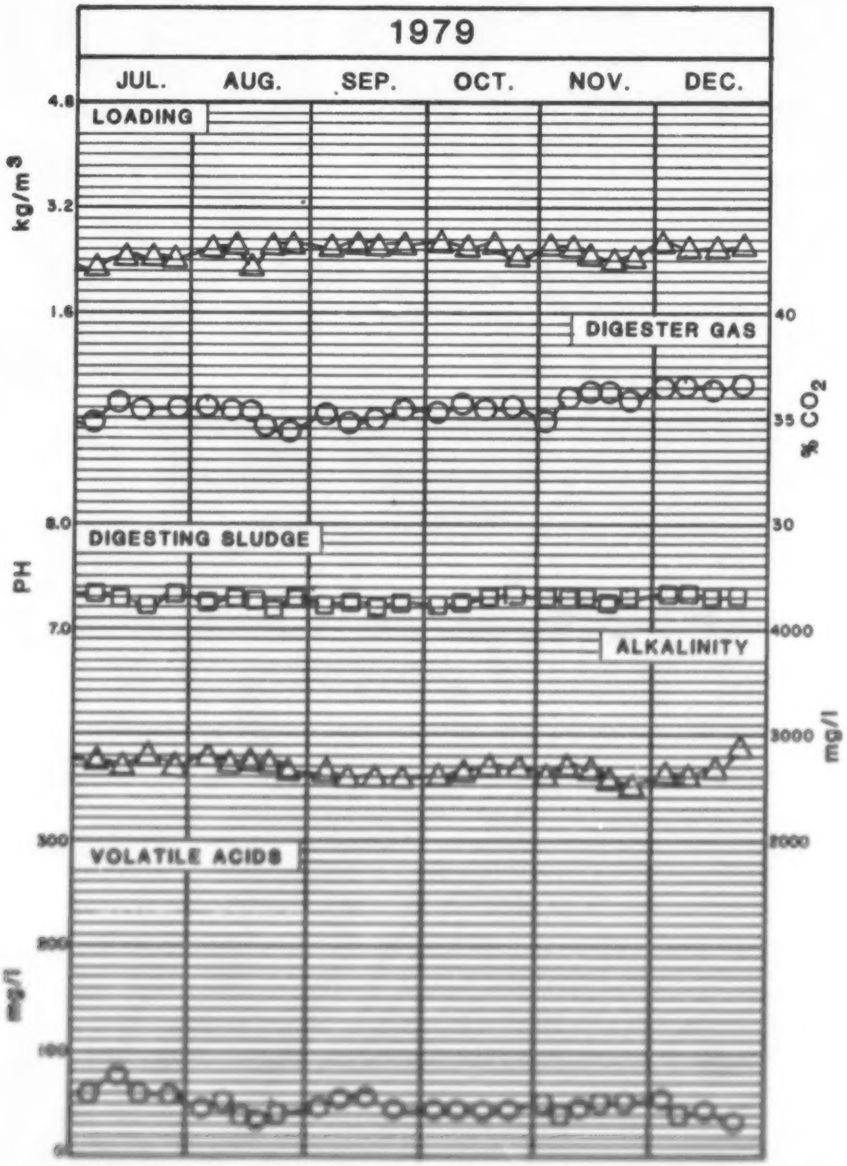


FIGURE 16. JWPCP Digestion Parameter, continued.

SLUDGE COMPOSTING: PROCESSES AND FUTURE DIRECTIONS

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INTRODUCTION

Composting is the biological oxidation of organic substrates in an aerobic environment (1). The dominant organisms are molds and thermophilic bacteria. Because living organisms grow on the organic substrate, it must contain the nitrogen, phosphorus, and potassium essential for growth as well as certain trace elements. The process is remarkably insensitive to operating conditions, and can take place in static piles that are only occasionally or never overturned for aeration as well as in frequently mixed windrows. A characteristic of composting is the generation of temperatures higher than ambient. Temperatures commonly reach thermophilic conditions (50-70 °C). This creates a substantial advantage for composting when treating sanitary wastes, because these temperatures destroy pathogens.

Composting of agricultural and animal wastes to produce a humus-like product is a process that has been utilized by man from antiquity (2). As anyone who has composted garden wastes is aware, the process transforms a bulky nuisance into a reduced mass of a valuable soil conditioner. As concern for environmentally and aesthetically acceptable disposal has developed in the world community, interest has grown in applying this desirable process to waste disposal. Composting primarily directed at converting sewage sludge into a useful or easily disposed product was unknown in the United States about fifteen years ago. Since then, development of sludge composting has been rapid. Already several full-scale operations are underway and many more are in the design stage. The process is probably receiving more attention and serious consideration than any other treatment or conversion process developed in recent years.

The objective of this presentation is to review the state of the art of sludge composting in the United States, consider the impact of developments in Europe, and describe EPA's continuing activities to develop the technology.

Composting of Sludge and Solid Waste

In the period from about 1960 to 1970, there was strong interest in composting of solid waste in the United States and in Europe (3). The process was combined with resource recovery of metals and glass from the solid waste. Sewage sludge, raw or digested, was added when available to provide nutrients. Virtually all of these enterprises in the United States have ceased operation. Their failure is attributed to the lack of a market for the product. The experience in Europe has been similar.

The failure of these processes has provided guidance for the development of composting processes for sludge. Solid waste is plentiful. Its abundance is one of the reasons why composting of solid waste has been unsuccessful. The large quantities of compost produced quickly saturate local markets and the price of the product falls.

Another problem has been the poor quality of the composts from solid waste. They have contained materials (glass, plastic, colored thread) that are inert and identifiable. These substances remain after biodegradable portions disappear and are aesthetically undesirable. In our development efforts, we have not encouraged work with substances other than sludge that would degrade the value fo the product.

Another competitive handicap for solid waste has been the relatively low cost of alternative means of disposal. The composts had to sell for a good price to offset the cost of composting. Unlike solid waste, processes to convert sludge to innocuous forms, to use it, or dispose of it are all expensive. Thus, reasonable costs can be borne for sludge composting without the need to recover most of the processing costs by sale of the product.

Windrow Composting of Sludge

The traditional method of composting of agricultural products and solid waste has been to pile the material to be composted into long rows (commonly called windrows), roughly triangular in cross-section. This procedure has been followed in windrow composting of sludge (4).

Oxygen for the composting process reaches the interior of the windrow by natural circulation of air through the mass as well as by periodic mixing. For circulation to occur and to maintain aerobic conditions, the pile must be porous and of the proper moisture content. Sludge cake must be pre-mixed with previously made compost, leaves, sawdust, or the like to achieve a moisture content of about 60 percent. Besides accomplishing air exchange, mixing also makes certain that all material in the pile is in the interior at one time or another where high temperature will destroy pathogens. In the first few days after the windrow is formed, mixing will likely be required more often then later on; and about 5 to 20 "turnings" are needed during the 3 to 4 week composting period. Mixing is usually accomplished with a mobile unit specially designed for mixing and turning compost. The windrows are from 3-7 feet high by 7-15 feet at the base. There is no blanket of previously composted sludge either underneath or covering the windrow.

An outstandingly successful windrow composting operation has been carried out by the Los Angeles County Sanitation Districts (5). At their Figeroa Street plant, 100 dry tons per day of digested sludge is composted, using previously made compost as a moisture-absorbing and bulking agent.

Part of the reason for their success is the warm dry Los Angeles climate and the use of a relatively high solids sludge cake. Windrow composting may perform better in more inclement parts of the United States if the operation is protected from rain and if a moisture absorbing agent is used, preferably one that also composts and adds its energy to the system.

The Beltsville Aerated-Pile Method

The aerated-pile method for composting sludge was developed by staff of the U. S. Department of Agriculture's Science Education Administration at Beltsville (USDA-SEA), in a cooperative effort supported by the U. S. EPA. This method has been described in numerous publications (6,7) and will not be described in great detail here.

In this process, air is drawn through a deep pile comprised of carefully premixed sludge and a bulking agent (such as wood chips) by means of blowers and air pipes placed underneath the pile in a porous base such as wood chips. The pile is constructed of carefully premixed sludge and bulking agent. It is then covered with a blanket of unscreened previously composted material, and is not mixed again unless the mixture for some reason did not adequately compost. This configuration is diagrammed in Figure 1. An extended aerated static-pile configuration is recommended for large installations. Essentially, adjacent individual piles are built onto one another. This extended configuration reduces the need for cover material by as much as 50 percent and the need for composting pad space by 50 percent. Details of the method are presented elsewhere (7).

A blanket of unscreened composted sludge is used to insulate the piles so that all the parts of the composting sludge will obtain a temperature of 55 °C in the coldest zone for a minimum of 2 days. The compost blanket also screens out malodorous volatile compounds that may be released from the pile. Total height of static piles should not exceed 12 feet including the base material of the blanket.

A pile of screened composted sludge is needed to adsorb the odor that comes from the blower that draws air through the pile. Approximately one cubic yard of screened composted sludge is needed for every 4 dry tons of sludge in the static pile. This odor adsorbing pile will cause too much back pressure and will not adequately remove odors if the moisture content is in excess of approximately 50 percent.

Presently the most successful sewage sludge composting operations are both simple and flexible in design and operation and are based on the static-aerated pile and windrow techniques. In Table 1, locations where composting is successfully practiced in the U.S.A. are listed together with selected characteristics of operations.

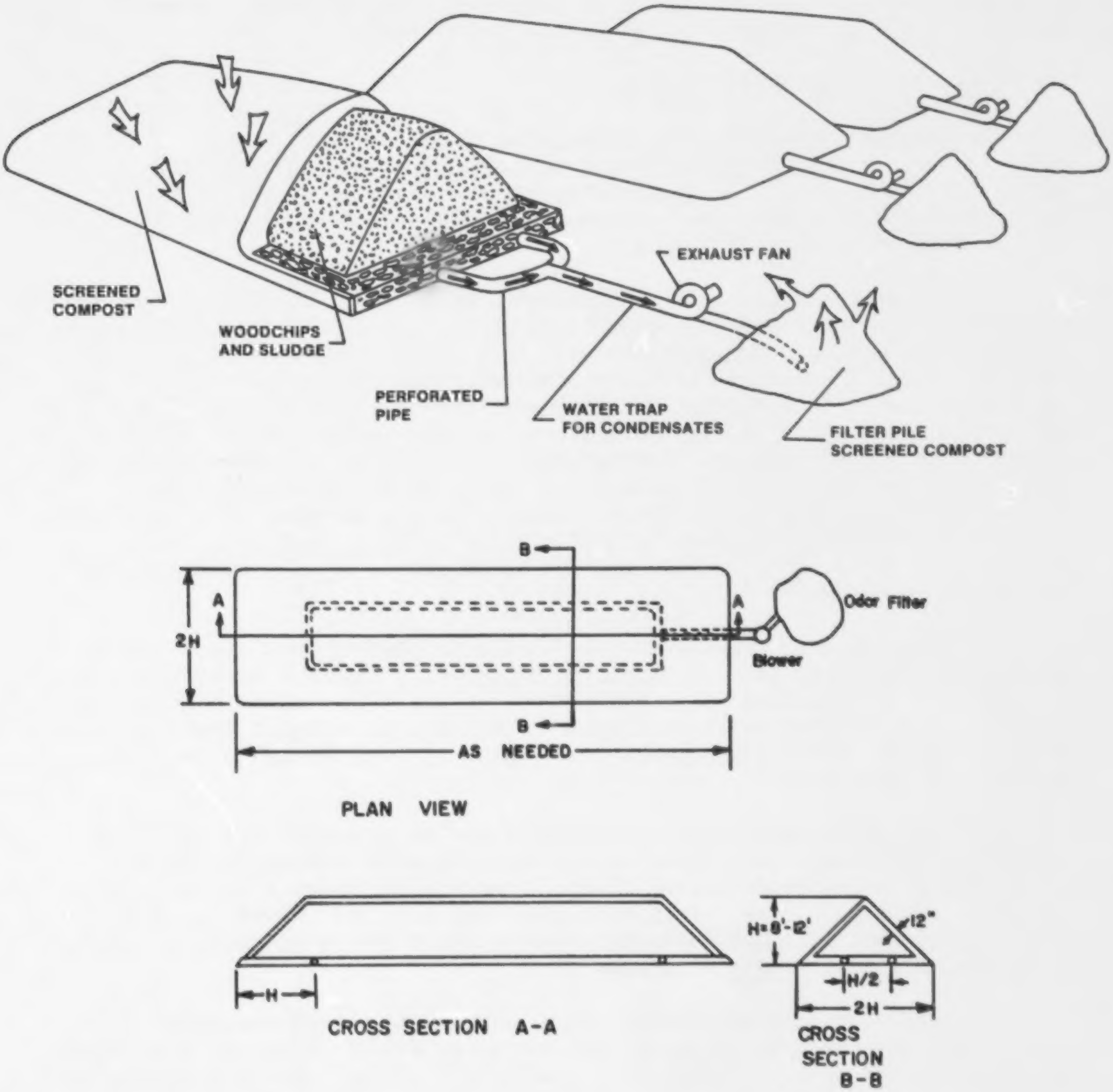

SCREENED COMPOST
WOODCHIPS AND SLUDGE
PERFORATED PIPE
WATER TRAP FOR CONDENSATES
EXHAUST FAN
FILTER PILE SCREENED COMPOST
B
A
2H
A
Odor Filter
Blower
B
AS NEEDED
PLAN VIEW
H
CROSS SECTION A-A
H=8'-12'
12"
H/2
2H
CROSS SECTION B-B

Figure 1. Beltsville Forced-Aeration System

TABLE 1. SELECTED CHARACTERISTICS OF SEWAGE SLUDGES AT STUDIED COMPOSTING FACILITIES

Location	Composting Mode(a)	Sludge(b)	Dewatering Mode	Solids Content %	Solids Content VS(c) %	Sludge Composition Lime %	Sludge Composition $FeCl_3$ %
Static Aerated Pile		Raw					
Washington, DC							
Beltsville, MD	E	P+S+CP	vacuum filter	17-24	55	25	8
Blue Plains	E	P+S+CP	vacuum filter	17-24	55	25	8
Camden, NJ	E	P	belt press	24-28	77	0	0
Bangor, ME		P	vacuum filter	22-28	60	1	0
Durham, NH		P	coil filter (CF)	20	-	10	5
Windsor, Ontario	E	P	centrifuge + CF vacuum filter	20-25	50	7	8
Windrow		Anaerobically Digested					
Los Angeles, CA		P	solid bowl/ basket centrifuges	23	75	0	0
Upper Occoguan, VA	A	AWT	filter press	40	40	35	5

(a) E - Extended static aerated pile
A - Aerated windrow
(b) P - Primary
S - Secondary
CP - Chemical precipitation of phosphate
AWT - P+S+CP+ ion exhange + filtration + C adsorption
(c) %VS - Percent of total solids present that are volatile

Guidance for Unconfined Sludge Composting Processes

Because of the importance the deep-pile and windrow composting systems are assuming in the United States, it is worthwhile to present some guidance based on experience gained thus far with these rapidly developing processes. The U. S. EPA's Office of Water Program Operations is preparing a technical bulletin on sewage sludge composting. This bulletin will provide guidance to Regional EPA Administrators, local authorities, and consulting engineers in designing and operating municipal sludge composting systems.

It is recognized that as more experience is gained the guidance will be revised. Nevertheless it usefully summarizes experience gained so far in the U.S.A. Some of the highlights from the draft bulletin are presented below.

For pathogen control it is recommended that the temperature of the composting sewage sludge should be at 55 degrees C or above for a minimum of 2 days to minimize pathogens. If in a windrow, the windrows should reach 55 degrees or above in the interior each time after 5 turnings during 15 of the 21-30 day composting period. (This suggestion is subject to change--the number of days at 55° may be reduced.)

Experience has shown that composting by the static pile or windrow techniques takes a minimum of 21 days to become adequately stabilized, for achievement of reasonably lowered moisture levels, and for pathogen reduction. The windrow operation may take a little longer time period than the static pile system. Furthermore, both composting operations may take a little longer during both wet and cold periods. One month storage of composted sludge for curing and additional pathogen reduction has generally been necessary after windrow composting and is desirable after static pile composting.

There are other conditions which are also of importance in achieving good composting. These are:

1. Cleanliness (minimizes malodor, dust, spread of *Aspergillus fumigatus* spores).
2. Proper blending with bulking agent to achieve moisture contents of 40 to 65 percent to permit uniform penetration of air.
3. Proper aeration.
4. Avoidance of opening composting piles during unfavorable meteorological conditions such as temperature inversion and very calm air. For example, this often occurs during summer in parts of the northeastern United States.
5. Keen sensitivity to the concerns of neighbors (screening with trees and shrubbery; planting of flowers and grass; and avoidance of noise, dust, and malodor).
6. Treatment with lime of particularly septic sludges or septages prior to exposure on the pad for mixing.

7. Back-up capability for temporary sludge disposal, such as by landfilling or trenching, when composting is not possible due to breakdown, bad weather, etc.

8. A several day sludge storage capacity before composting to avoid a necessity of operation on the wettest of days.

9. Enclose the screening operation to minimize dust.

10. Hard surface pad for active composting, preferably of concrete.

European Within-Vessel Composting Processes

Composting is an aerobic biological process and, as such, depends on pH, moisture, oxygen, and major and minor nutrients. All of these factors are needed to be optimized for optimum composting process. The composting operations in U.S.A. are being conducted mainly in the open air, and there is obviously no control over temperature, humidity, and precipitation. It has been claimed that by composting within an enclosed vessel the above parameters can be better controlled and the time period can be reduced from 21 days to one week or even shorter. Additionally, effective odor control, minimum labor, and low maintenance are among the advantages claimed for within-vessel composting.

A few years ago, reports were received that within-vessel sludge composting processes were receiving rapid acceptance in Europe. At this time, one manufacturer (BAV) initiated sales efforts in the United States. To evaluate these and other claims a review study of the status of European practices on sewage sludge composting was conducted by Battelle Laboratories under a contract with the U. S. Environmental Protection Agency (8). One of the objectives of this study was to identify the existing composting practices in Europe and compile information on operating modes, effectiveness, reliability, and costs.

Since the technology is relatively new and is evolving rapidly, a variety of sources, in addition to the literature, were used to obtain the necessary information. Discussions were held with composting process inventors, license holders, plant manufacturers, and plant operators. Representatives from Ministries on Environmental Affairs from France, Germany, Italy, The Netherlands, United Kingdom, Sweden, and Switzerland were also interviewed to obtain their evaluations and leads to new data on sewage sludge composting. In addition, site visits by Battelle-Geneva, Battelle-Frankfurt, and Battelle-Columbus scientists to selected operating plants were made.

Of the seven European countries surveyed for sewage sludge composting practice, West Germany is the center of activity with more than 30 commercial within-vessel plants. Sweden follows with 20 which are either in operation or in the planning and design stage; Switzerland has nine plants of which eight are the Dano rotating drum process; France has five; the United Kingdom has one; Italy and The Netherlands have none. These are plants which use sewage sludge as the predominant waste component. If municipal solid waste (MSW) intensive systems are also included, these numbers are much higher.

In Table 2, the major European composting processes were presented according to the process type.

TABLE 2. EUROPEAN SEWAGE SLUDGE COMPOSTING PROCESSES

Category	Process	Number of Operating Plants
Within Vessel[(a)]	BAV	19
	Carel Fouche Languepin	1
	Roediger/Fermenttechnik	1
	Schnorr Valve Cell	2
	Societe General D'assainissement et de Distribution (SGDA)	1
	Triga	2
	Weiss	3
Windrow	BIO-Manure	2
	Hazemag	-
	PLM	-
Rotating Drum[(a)]	Buhler	9
	Dano	-
	HKS	2
Pressed Brick	Brikollare	2
Fermentation Cells	Prat	1

(a) These processes usually include a final windrow cure.

In the following paragraphs, six European "within-vessel" processes are briefly described. The information includes the unit sizes, operating history, and the plant locations.

Carel Fouche Languepin Process - The Carel Fouche Languepin (CFL) Company has been building composting plants since 1965. Fermentation takes place in cells arranged in blocks comprising five levels. A vertical set of five cells represents one fermentation unit (or digester). Two blowers per digester can inject or draw air from each of the five cells fitted with air distribution ducts. The top cell is charged with the freshly pulverized MSW. Sludge can be added to the MSW before feeding. The fermentation mass passes vertically down the digester in five steps, and communication between cells is achieved by tilting floor gratings. The residence time in the digester is approximately a week. The only CFL plant which processes MSW and sludge is the Montbeliard plant which has been in operation since mid-1977. The daily capacity is 150 t/d or MSW and 5 t/d of sludge with a water content of 80 to 85 percent.

Development of a new digester--CFL is just beginning a research program aimed at the development of a new digester for the fermentation of sludge. This research is sponsored by the French Environmental Ministry. The pilot digester will process 1 m^3/d of sludge mixed with carbon sources and/or minerals and/or polystyrene spheres. The aim of this research is to obtain a product similar to a peat because the French importation of peat is expensive.

BAV Process - Biologische Abfallverwertungsges, mbH & Company (BAV) has constructed at least 19 plants in West Germany within the past several years and has at least five additional plants being designed or under construction. Most BAV reactors are designed for capacities equivalent to populations of 30,000. However, BAV systems have been designed to service communities of 10,000 and to service cities of 110,000.

The BAV process has evolved from the general method that was patented by F. Kneer (German patent No. 2253009, June 19, 1975). Essentially, the process consists of aerating a mixture of dewatered primary sewage sludge, sawdust, and cured compost in an open-top, cylindrical vessel. The dewatered sludge, sawdust, and cured compost mixture slowly descends, by gravity, through the vessel's length in about 12 days. The descending mixture is aerated by compressed air which is distributed at the base of the vessel. The composting rate can be controlled, within limits, by the aeration rate. A diagram of the reactor vessel and its components is presented in Figure 2.

A conveying screw is used to remove product at the base of vessel mechanism. This screw rotates around the vessel's radius and brings the compost product towards a center exit port. The product, after within-vessel composting is placed in windrows for a curing period.

SGAD Process - In 1975 the "Societe Generale D'assainissement et de Distribution" (SGAD) built a plant in Salon de Provence under a BAV license to process sewage sludge and municipal solid wastes. Initial problems with the extraction screw conveyor led to key modifications and a proprietary design. No other plants have been built in recent years.

Salon de Provence plant - SGAD operates this plant which processes the municipal solid wastes and the sludge for a population equivalent of 60,000. The MSW is ground and screened. The larger fraction is fermented. The latter fraction is mixed with sludge having a water content of 94 percent. This mixture is then fed into two reactors of 600 m^3. There is a buffer storage between the mixer and the reactors. The temperatures at different locations and the CO_2 content of the exhaust gases are recorded. The residence time in the reactor is about 15 days. The mixture is then removed and sent to the curing piles. The curing is carried out in 2 m high piles approximately 10 weeks without turning.

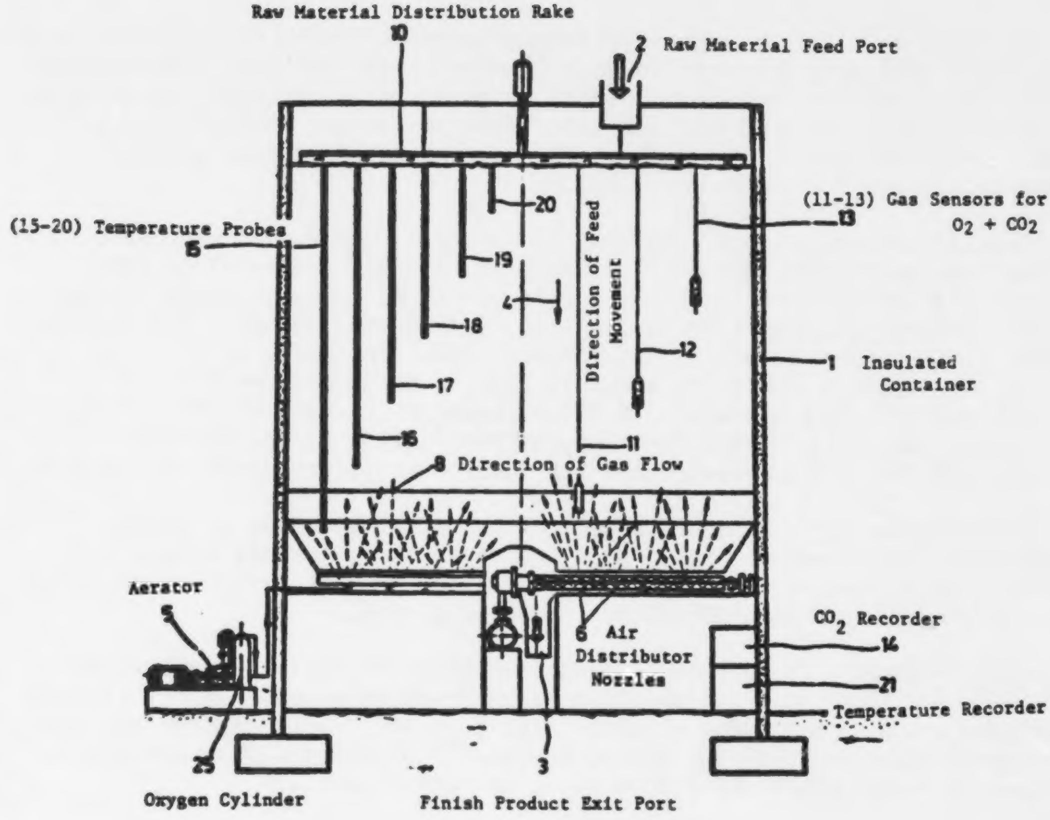

Raw Material Distribution Rake
10
2 Raw Material Feed Port
(11-13) Gas Sensors for
13
O2 + CO2
(15-20) Temperature Probes
15
20
19
4
18
17
16
Direction of Feed Movement
12
11
1 Insulated Container
8 Direction of Gas Flow
Aerator
5
6 Air Distributor Nozzles
CO2 Recorder
14
21
Temperature Recorder
25
Oxygen Cylinder
3
Finish Product Exit Port

Figure 2. BAV Composting Vessel

Schnorr Valve-Cell Process - There are two operating plants in West Germany based upon the Schnorr Valve-Cell Process, one at Denkendorf and the other at Rastatt. A third plant is under construction at Gaggenau. The manufacturer is Fa. Dambach Industrieaniagen GmbH.

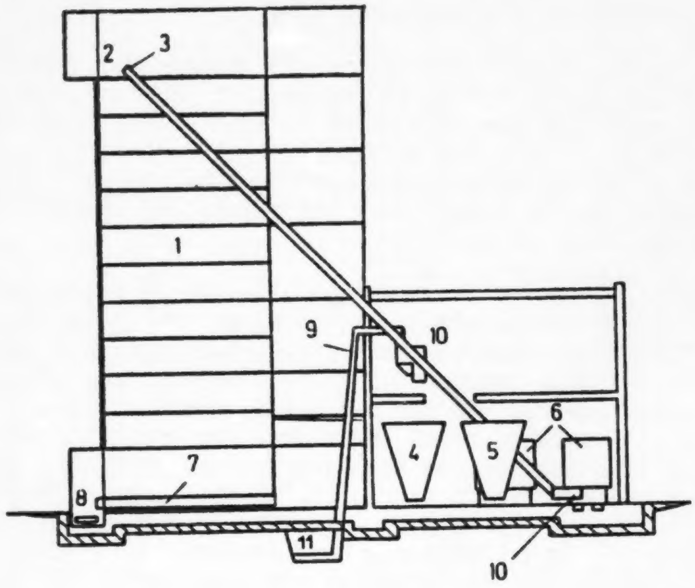
The fermenting towers used in the valve-cell process shown in Figure 3 are 10 m long, 4 m wide, and 15 m high and consist of 10 tiers, one above the other. The floor of each tier consists of perforated valves which can be opened and closed hydraulically. The fermentation mixture is fresh sludge with a solid content of 22-25 percent plus ground-up bark with a diameter of 5-10 mm and is fed into the top tier. When the valves are opened, the material falls onto the floor below.

At the start of the process, the towers are initially filled with an equal mixture of dewatered sludge and ground-up bark. After the process is established, a portion of finished compost is recycled back into the system by mixing it with fresh sludge and bark in a 2:2:1 proportion. Thus, the quantity of ground bark required for the process is reduced. The mixer, a scrape conveyor track, is located below the sludge and recycle compost containers. The product (feedstock) from the mixer passes through a set of rubber rollers before being conveyed to the top of the tower. The purpose of running the feedstock through the rubber rollers is to further loosen the material, by breaking any lumps which may have formed. This tends to keep the feedstock fluffy, which thereby increases the opportunity for aerobic composting.

Every third day, the content of the tower is let down one tier at a time by opening the floor valves so that a residence time of 30 days results. Every second tier is aerated from the front and the alternate tiers from the back. Directing the air in this way ensures that it flows through the fermentation material. The fermentation mixture is spread out in layers of 1 m deep in each tier, and as it drops from tier to tier, further mixing and loosening occurs. Temperatures of 70 °C and above are reached during a two-week fermentation period; thus, the sewage sludge is made completely hygienic. The temperature is utilized to control air flow to the process. Temperature probes are attached at three places in the fermentation towers.

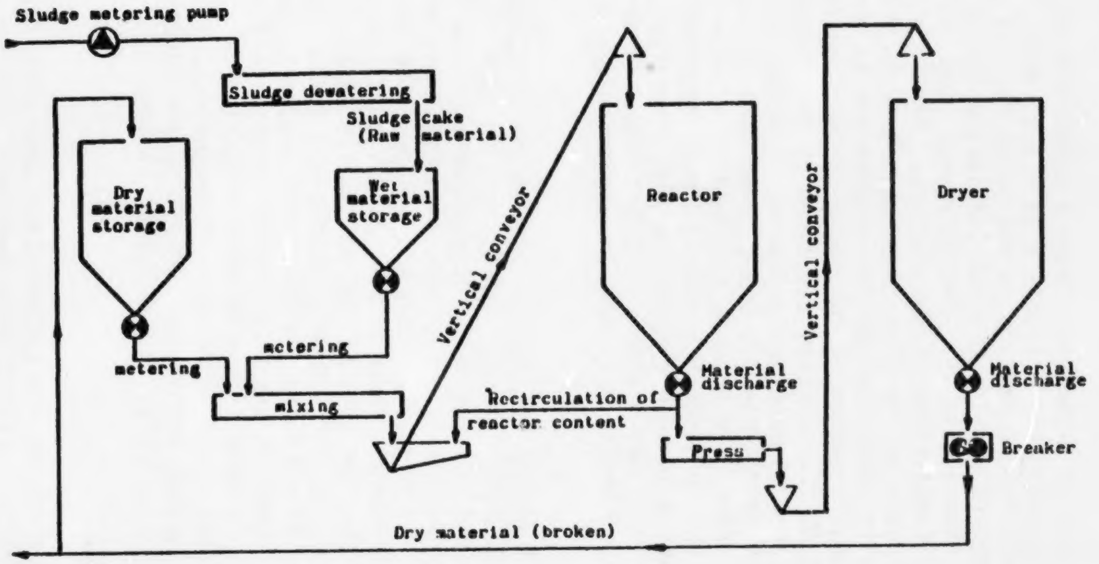
Roediger/Fermenttechnik Process - This process was developed by Professor Baader (Institute for Agricultural Technique, Brunswick) for animal wastes and has been adapted for sewage sludge (Figure 4). Fa. Wilhelm Roediger Industriehafen has constructed only one demonstration plant which is located at Mittleres Wutachtal, West Germany.

Previously dewatered sludge is combined with dried returns in a double-shaft mixer to obtain a mixture with approximately 50 percent water content. The mixture is fed into the reactor by a vertical conveyor. Aeration in the fermentation reactor is achieved by having floor grating. At certain intervals during the fermentation process, the grating is set in motion and the material falls through a catch funnel into a screw conveyor fixed below it. The material is then fed back into the reactor via the vertical conveyor. Air is blown through the grating at such a high rate that excess oxygen is



1 biocell reactor
2 distributor
3 conveyor
4 sludge container
5 returns container
6 coagulant container
7 discharge conveyor
8 dump conveyor
9 sawdust conveyor
10 pumps
11 sawdust bunker

Figure 3. Schnorr Biocell System


Sludge metering pump
Sludge dewatering
Sludge cake
(Raw material)
Dry material storage
Wet material storage
metering
metering
mixing
Vertical conveyor
Reactor
Recirculation of reactor content
Material discharge
Press
Vertical conveyor
Dryer
Material discharge
Breaker
Dry material (broken)

Process flow sheet of the FERMENTECHNIK System

FERMENTECHNIK
for Environmental Protection

EURAMCA INC.

Figure 4. Roediger and Fermentechnik Method

supplied. The fermentation process reaches a disinfection temperature of approximately 72-74 °C within 24-36 hours. The circulation process mentioned above ensures that all material passes through this high-temperature zone several times during the fermentation period. The material is then formed into pellets and dried.

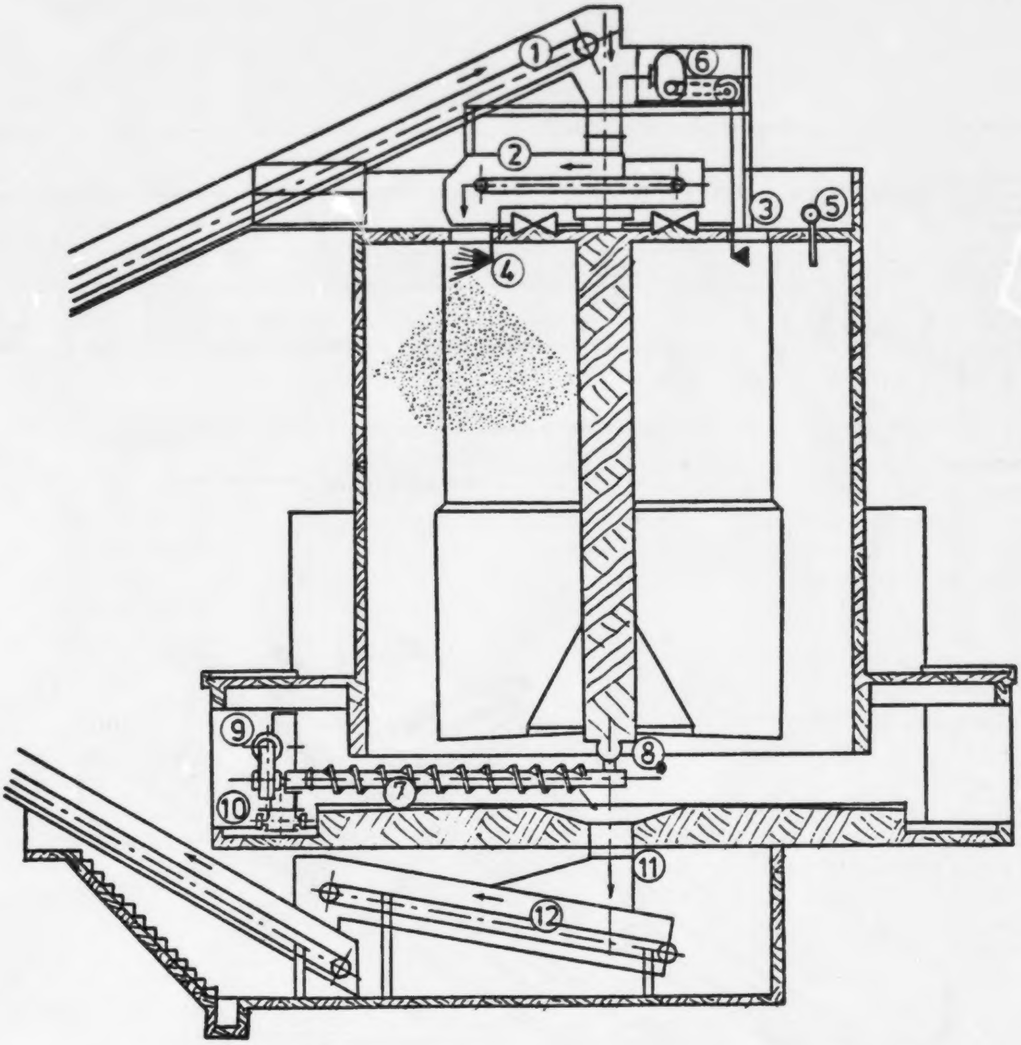
Triga Process - The reactor is a concrete tower divided into four compartments and is called a "Hygienisator." Figure 5 is a diagram of the Triga composting reactor. The sludge refuse mixture is fed in at the top of the tower. A concrete wall is used to provide thermal insulation and to assure that design temperature is achieved quickly. The residence time varies from 4 to 10 days according to the composition of the mixture. Air is pulled out of the top of the tower to produce an air flow from the bottom to the top of the tower. The air-flow rate is controlled by the temperature in the bed. At the start of the process, the air flow is regulated so that the temperature in the bed remains at 70-80 °C. A screw extractor at the bottom is used to remove the product. The screw is externally mounted to permit better access. The fermentation mass is recycled to avoid compaction at the bottom of the tower. After the digestion, the product is cured to achieve additional stabilization. Since this process draws air through the reactor, control of potential odor problem is much simpler, at the discharge of the blower. At the beginning of 1978, two sludge-only plants were in operation in France, although plants based on MSW have been in operation for about 10 years.

Summary Comments on the European Processes - The review of European processes and a detailed examination of one of them revealed some interesting information. It is without question that the processes work. They produce a compost product which should find better acceptance than solid waste compost. Municipalities are adopting the processes. They all appear to be expensive, and a detailed analysis of one process showed total annual costs (including amortization) to be several times the cost of the Beltsville Aerated Pile method. Technical problems exist. For example, conditions are unlikely to be uniform across the cross-section of the within-vessel reactors. Systems that mix the mixture or recirculate it are more likely to produce a uniform product. The processes generally show little economy of scale, so costs per ton do not decrease much as plant scale increases. Nevertheless, it is evident that within-vessel processes for sludge composting are successful and worthy of consideration by municipalities in the United States.

Within-Vessel Systems in U.S.A.

Several solid waste-sludge composting systems have been available in the United States for many years, but no systems were available specifically designed for sludge. Recently some United States manufacturers have adapted their processes to sludge composting, and some European manufacturers are attempting to penetrate the U. S. market.

Metro-Waste - The Metro Waste composting system, which is marketed by Resource Conversion Systems, Inc., is fundamentally a forced aeration static pile system housed in an elongated rectangular container (Figures 6 and 7). The loading, mixing, and unloading of sludge and bulking agent are



1 - 2 - 12 belt conveyors
3 feed port
4 water spraying nozzles
5 temperature probe
6 aerator
7 screw extractor
8 axis
9 trolley
10 rolling path
11 finish product exit port
12 finish product conveyor

Figure 5. Triga Composting Reactor

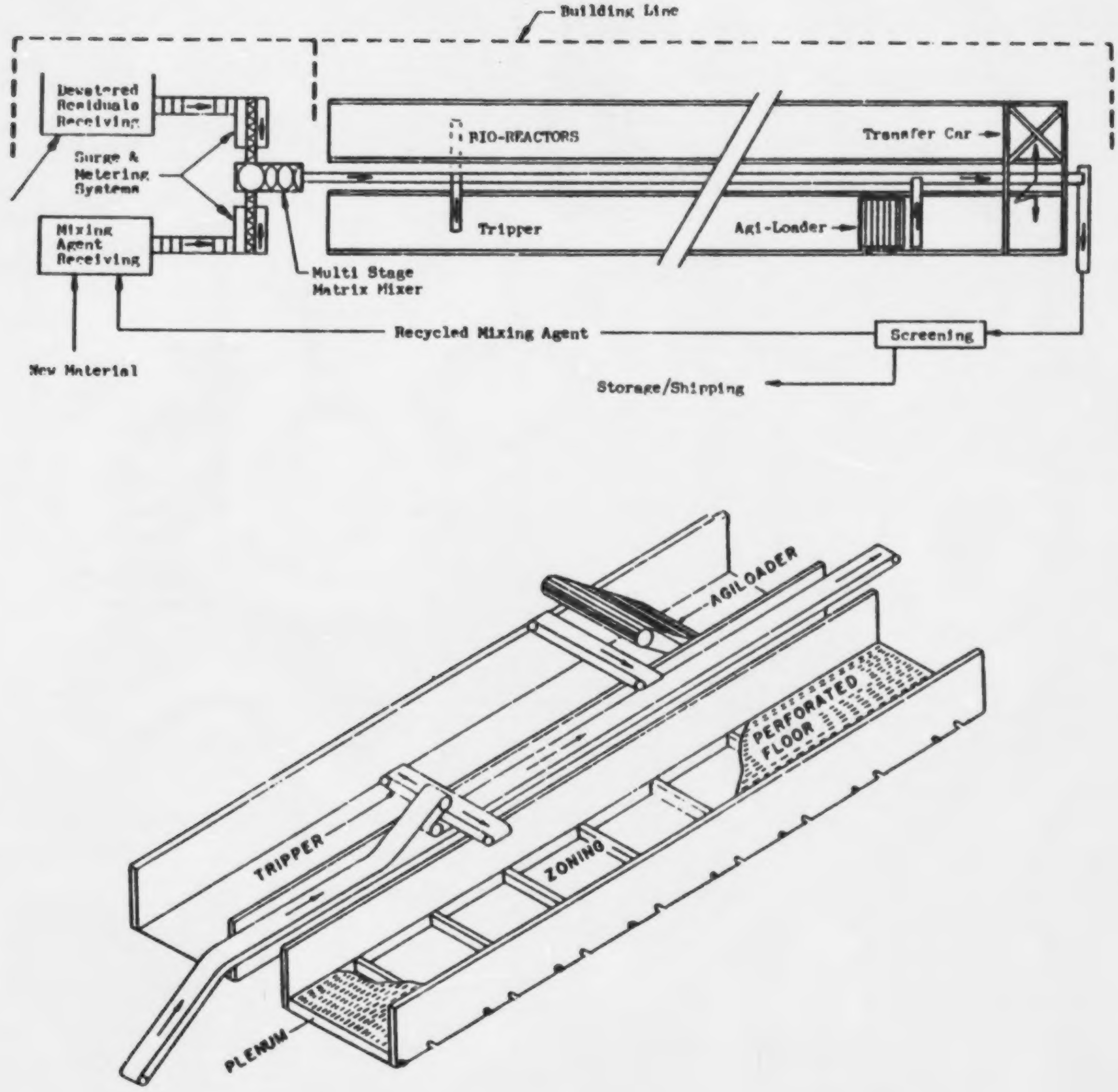

Building Line
Dewatered Residuals Receiving
Surge & Metering Systems
Mixing Agent Receiving
Multi Stage Matrix Mixer
BIO-REACTORS
Tripper
Agi-Loader
Transfer Car
Recycled Mixing Agent
Screening
New Material
Storage/Shipping
AGILOADER
PERFORATED FLOOR
TRIPPER
ZONING
PLENUM

Figure 6. Metro-Waste Bioreactor

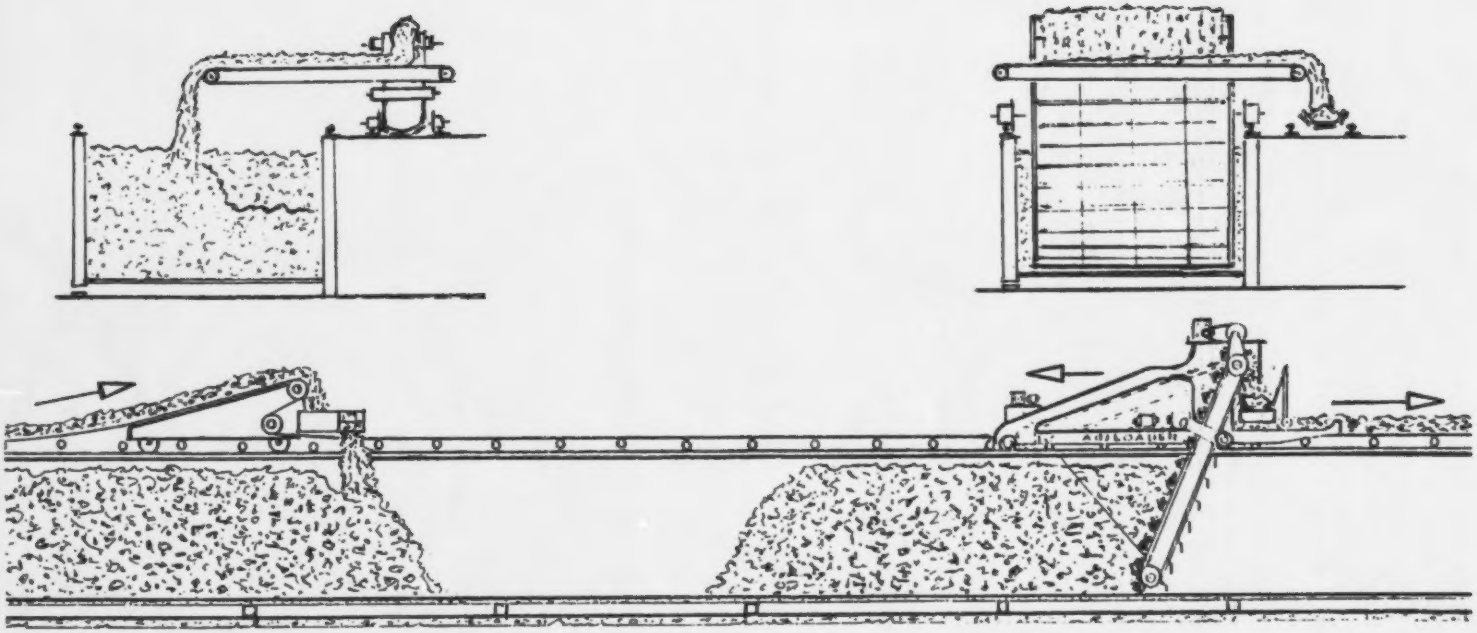


Figure 7. Loading and Unloading of Metro-Waste Bioreactor

accomplished mechanically by a conveyor system and an "agiloader" (9). This system was originally developed in 1964 for municipal solid waste at Largo, Florida, and later used for mixture of sewage sludge and solid waste. However, because of the lack of a market for composted solid waste, this and two other operations were abandoned. Presently, Resource Conversion Systems, Inc., is actively promoting this system for sludge application.

Paygro System - The Paygro system was originally based on a metro-waste system. Modifications were made and the improvements were patented by Paygro, Inc. Since 1972, bark and cow manure were composted and marketed as potting soil. Composting facilities, which occupy about 40,000 square feet of enclosed reactors, are located at South Charleston, Ohio. During the last year, Paygro, through their manufacturing agent, the Henry P. Thomson Company, have been experimenting with municipal sludge. Based on the preliminary results, they expect to construct full-scale facilities for a municipality during the next year.

Fermentechnick-Euramca - This European-developed system is being actively promoted by Euramca, Inc., of Addison, Illinois. The system was described above under European systems. Euramco, Inc., has a portable 5 m^3 test unit which can be used for preliminary studies. The unit was experimented on at Los Angeles and other places but as yet has not been adopted for full-scale operation.

BAV System - For a period, the BAV system was actively marketed in the U. S. and Canada. There seems to be little marketing effort now. BAV appears to be directing its efforts to a new approach based on passage of the mixture to be composted through a horizontal reactor.

Future Activities Concerning Sewage Sludge Composting

The several activities planned or in progress by EPA's Municipal Environmental Research Laboratory (MERL) are listed below.

1. Within-Vessel Composting. MERL is continuing evaluating European systems under a contract with Camp, Dresser, and McKee, a Boston consulting firm. They will be concentrating mainly on existing BAV bioreactors in Germany and a recently introduced tunnel reactor by BAV.

2. Metro-Waste and Paygro Systems. Paygro System is the only within-vessel composting system (now composting mainly cow manure) operating on a large scale that seems adaptable to a compost feed mainly comprised of sewage sludge. For the summer of 1980, it is planned to evaluate the system using sewage sludge from Columbus, Ohio.

3. Indices for Composting. To determine the progress of composting, parameters conventionally used to indicate degree of sewage treatment or sludge stabilization are normally used. These parameters are biological oxygen demand (BOD), chemical oxygen demand (COD), caloric content of sludge, volatile solids content, carbon to nitrogen ratio, etc. Although they successfully indicate, for example, the progress of anaerobic digestion, their suitability for composting has not been documented. Through cooperative

agreements with municipalities or universities, investigations will be conducted to identify the most significant parameters, so that the extent of composting can be evaluated, and the performance of various composting systems can be compared.

4. Optimize Static-Pile Composting. A project is now underway aimed at evaluation and development of temperature control strategies to maximize organic matter degradation in aerated static-pile composting. This is directed by Drs. Melvin Finstein and John Cirello of Rutgers University. The main objective of this proposal is to demonstrate that the composting process can be hastened by using temperature control as a primary process control parameter. Benefits expected from the successful completion of this demonstration are: a) more rapid stabilization, thereby decreasing space and facility requirements; b) insuring a well-stabilized end-product; and c) minimizing the amount of process residue needing final disposition.

The above objectives will be accomplished by evaluating various temperature control strategies while being mindful of the interrelationships among temperature, moisture, and aeration. The field experiments will be performed at the Camden, New Jersey, treatment plant site, using compost piles specifically operated for this research.

5. Utilization of Compost. An extremely important subject, which is outside the scope of this presentation, is the utilization of compost. The composition of compost from sludge reflects the composition of the sludge from which it is derived. Compost made from a sludge containing some toxic metals must be used with caution, or possibly should not be made at all. An interagency agreement is continuing with the USDA-SEA to investigate the uptake of metals in crops grown on soil amended with compost made from sludge.

6. Evaluation of New Technology. It is expected that Innovative and Alternative Technology Program being conducted by EPA's Construction Grants Division will result in construction of plants using new approaches to composting, including within-vessel systems. MERL plans to grant funds to municipalities to document the performance of these systems and publish evaluations. At this time, no grants have been made but at least one application is expected within a year.

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